Investigation of Dynamic Capacity Analysis for Wave Loading Conditions

<u>Joint Industry Project</u>

by

<u>**PMB Engineering, Inc.**</u>

San Francisco, CA 94111

<u>**January 1994**</u>

Further Investigation into Capacity Analysis Procedures

Investigation of Dynamic Capacity Analysis For Wave Loading Conditions

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Contents

ILLUSTRATIONS

Figures

Figures

Figure

Figure

Tables

Executive Summary

This project has studied techniques for assessment of existing offshore platforms exposed to wave and seismic loadings. For wave loadings, the emphasis was a rigorous, in-depth analytical investigation to determine if the standard industry practice of performing a "static pushover" analysis provides an accurate measure of platform capacity. The wave loading portion of the project is contained in this report. For seismic loadings, the emphasis was development of a document describing the current "state-of-practice" techniques for seismic analysis of offshore platforms, with emphasis on reassessment. The seismic portion of the project is contained in a separate report.

The approach used for the wave loadings study was to select a typical Gulf of Mexico platform, perform a static pushover to determine the platform's "static" capacity, and then compare it to the "dynamic" capacity determined by a more complex series of structural analyses more representative of actual platform and loading conditions. The more complex analyses were labeled "dynamic" since they account for dynamic related issues associated with platform mass, damping and time-of-loading effects not contained in the static pushover analysis.

The platform studied was a 271 ft. water depth, eight leg platform of typical 1960's vintage for the Gulf of Mexico. Figure ES-1 shows the platform configuration. This platform was selected since it is typical of many existing Gulf of Mexico platforms and is in a water depth where dynamics may have an impact on the platform capacity. The platform was also previously studied in PMB's AIM (Assess, Maintain and Inspect) Phase III project [PMB, 1988] which provided the project with some initial insight on expected performance of the platform.

The project initiated with a literature search to identify issues related with static and dynamic structural analysis that should be incorporated in the project. Special emphasis was made to identify technology in fields other than offshore engineering that may be useful. Little additional information was located that was not already known to the participants and the PMB project team. Note that this was a limited literature search and that a more extensive effort may have identified some new technologies.

Both two-dimensional and three-dimensional CAP (Capacity Analysis Program – PMB's nonlinear analysis program) computer models were developed for the platform. The two-dimensional models, which were carefully calibrated against the three-dimensional models, were used for a majority of the analysis work since they are quicker to analyze, with results being easier to interpret. The three-dimensional model was used as a check against several of the two-dimensional analyses.

A static pushover analysis was performed to obtain the static platform capacity for comparison with the dynamic capacity obtained from later analysis. Both wave-below and wave-in-the-deck loading conditions were considered. Wave-in-the-deck loads were based upon a simplified procedure developed by the API Task Group 92-5 on Assessment of Existing Platforms for Suitability of Service. Figure ES-2 shows results for a typical static pushover, base case configuration, with waves in the deck. The analysis indicates that the platform will fail for a 73 ft wave (or larger).

A comparison was also made between wave loads determined by the API RP 2A 17th edition wave load recipe [API, 1987] and the API RP 2A 20th edition recipe [API, 1993]. The 20th edition recipe global loads on the platform were found to be 60 to 70 percent higher than the 17th edition recipe, for loading in the same direction without accounting for platform orientation. The difference would reduce when platform orientation is considered as allowed by the 20th edition.

Dynamic analysis consisted of regular waves, synthetic irregular multiple waves and measured irregular multiple waves (from Hurricane Camille).

Figure ES-3 shows time history motions of the platform deck for the base case configuration, with waves in-the-deck, with loading from multiple waves. The member failure sequence is also indicated (P2, P1, L7, etc.). This is expected to be a very realistic wave loading condition. This analysis (and the other multiple wave analyses) indicated that the platform will fail for a 75 ft wave, which is approximately the same result as for the static pushover. Table ES-1 summarizes results for the base case platform configuration.

The base case configuration was varied in order to investigate the effects of platform mass, stiffness and framing schemes. The platform mass (first deck then jacket) was increased for some cases and the platform stiffness (first jacket then deck leg) was decreased for other cases until dynamic effects were observed. Table ES-2 summarizes some of these results, indicating that there is approximately a 10 to 15 percent dynamic effect (dynamic capacity greater than static capacity) for two times or greater differences with the base case configurations. However, decks of this size (two to three times base case or 5,000 to 7,500 tons) begin to be outside of the range of typical Gulf of Mexico offshore operations.

The framing scheme was changed from the base case k-braced, to diagonal-braced and to x-braced. Both the diagonal-braced and the x-braced indicated similar dynamic effects as for the base case condition (i.e. some dynamic influence at higher mass or lower stiffness). Table ES-3 summarizes some of these results of the framing scheme comparisons.

Overall conclusions of the project are as follows:

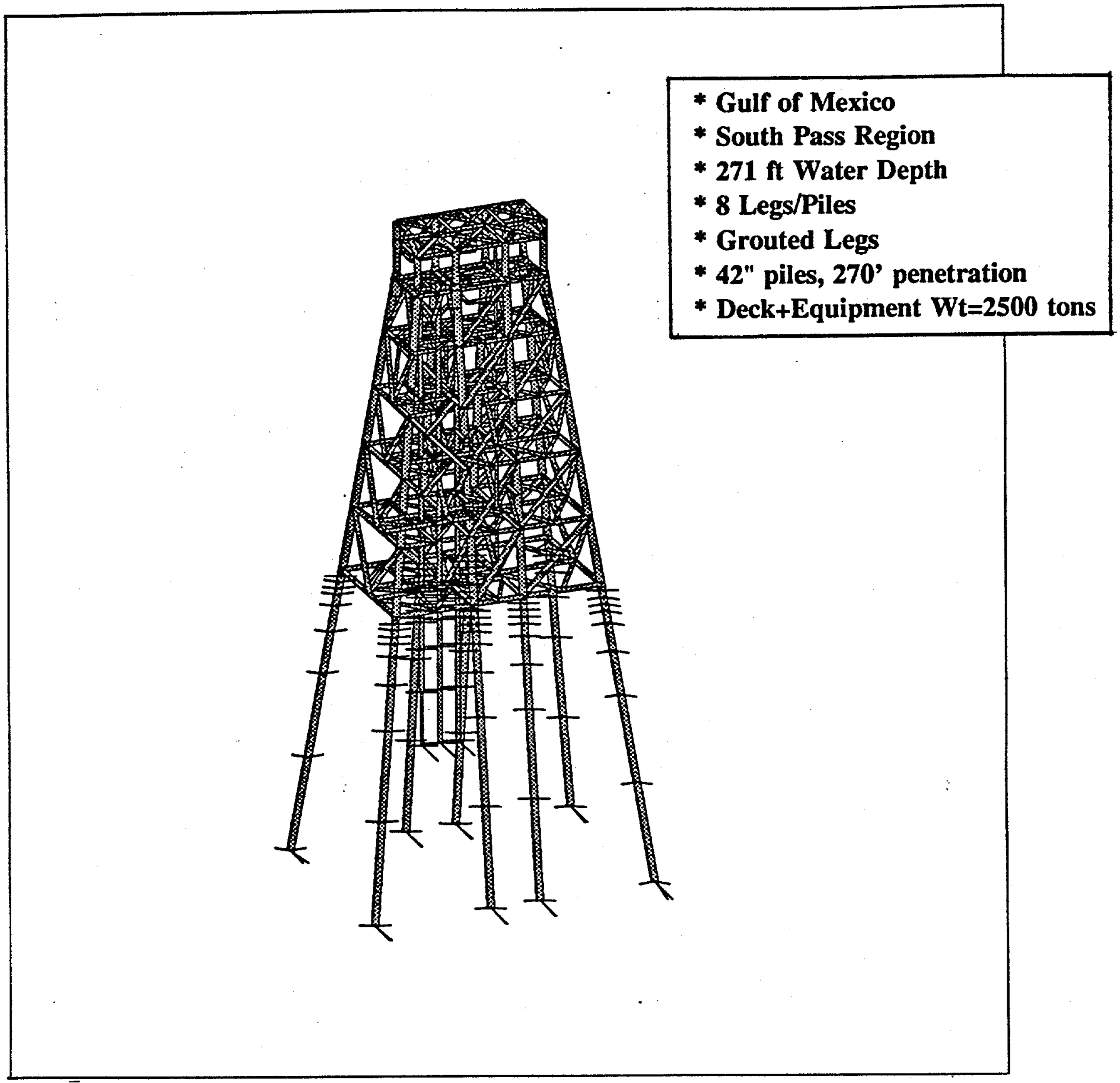
- For the platform configurations studied in the project, both inertia force and cyclic loading dynamic effects were observed. Large inertia force amplification was observed in the portal frame for the impulsive deck load. Cyclic loading behavior accounted for the observation that the structure did not fail during the critical wave cycle, but in a subsequent smaller wave cycle. However, for the structural configurations studied these observed dynamic effects did not significantly change the critical wave height identified by the more conventional static pushover method.

- For exceptionally heavy decks and exceptionally weak structures (e.g., heavily corroded) where there is wave loading in the deck, the platform capacity determined by dynamic analysis was higher than that determined by the static pushover. The increase ranged from 10 to 15 percent. These situations (heavy deck or weak structure) may not be typical of Gulf of Mexico operations. This increase was not apparent for wave-below-deck loading conditions.

- Changes in configuration of the platform bracing scheme (k-brace versus diagonal-brace versus x-brace) had little effect on the above conclusions (i.e. all exhibit similar behavior)

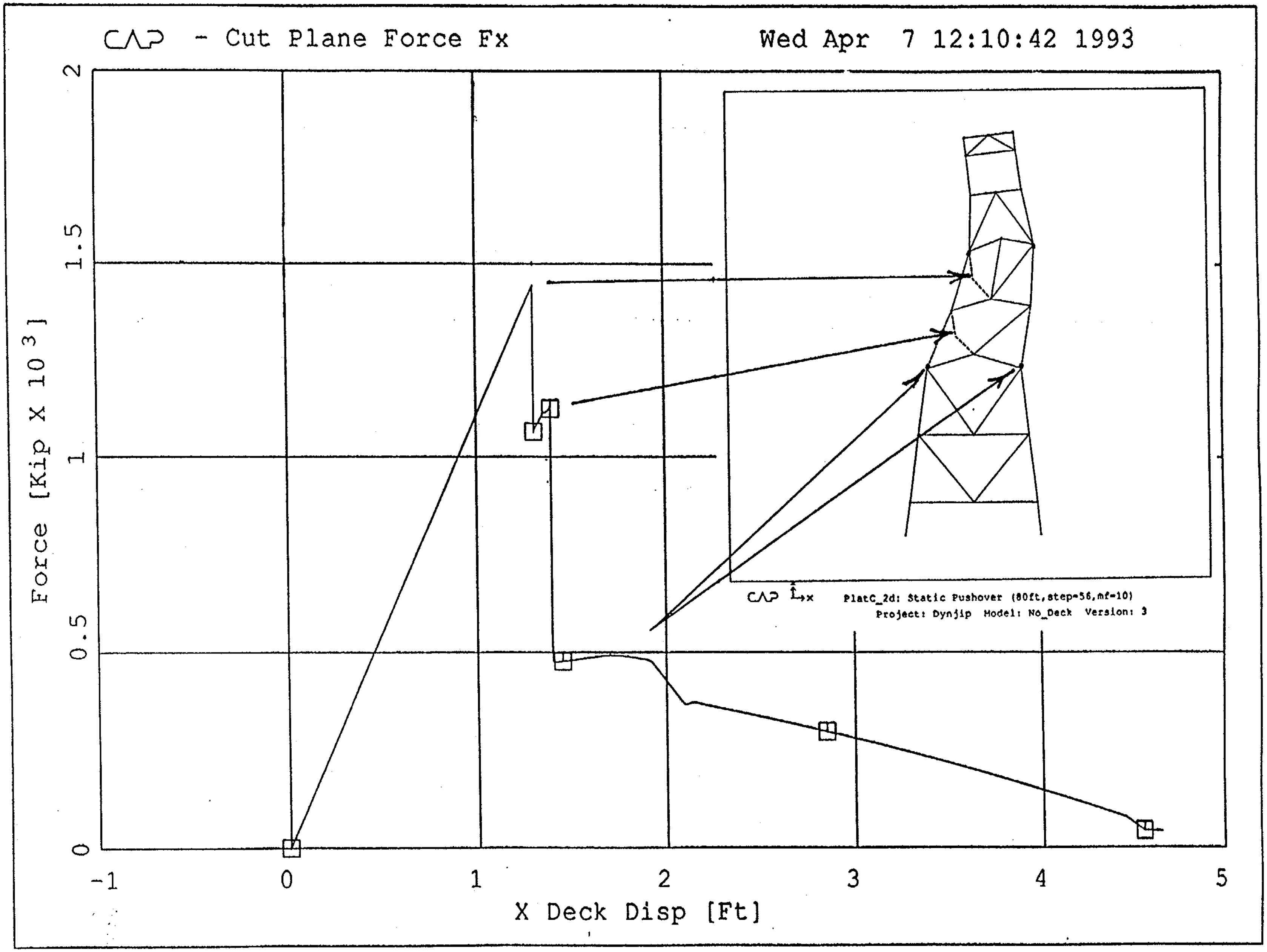
Overall recommendations for the project are as follows:

- It is recommended that the industry standard static pushover still be used as a basis for estimating platform capacity. For cases where there is wave-in-the-deck, it is recommended that further investigation be taken to determine the potential DAF of the wave-in-deck loads, as outlined in Section 9. This procedure reviews the platform static characteristics developed from the static pushover and a few dynamic properties (global and portal natural periods) to assure that the particular platform does not possess some unusual characteristics sensitive to dynamics.



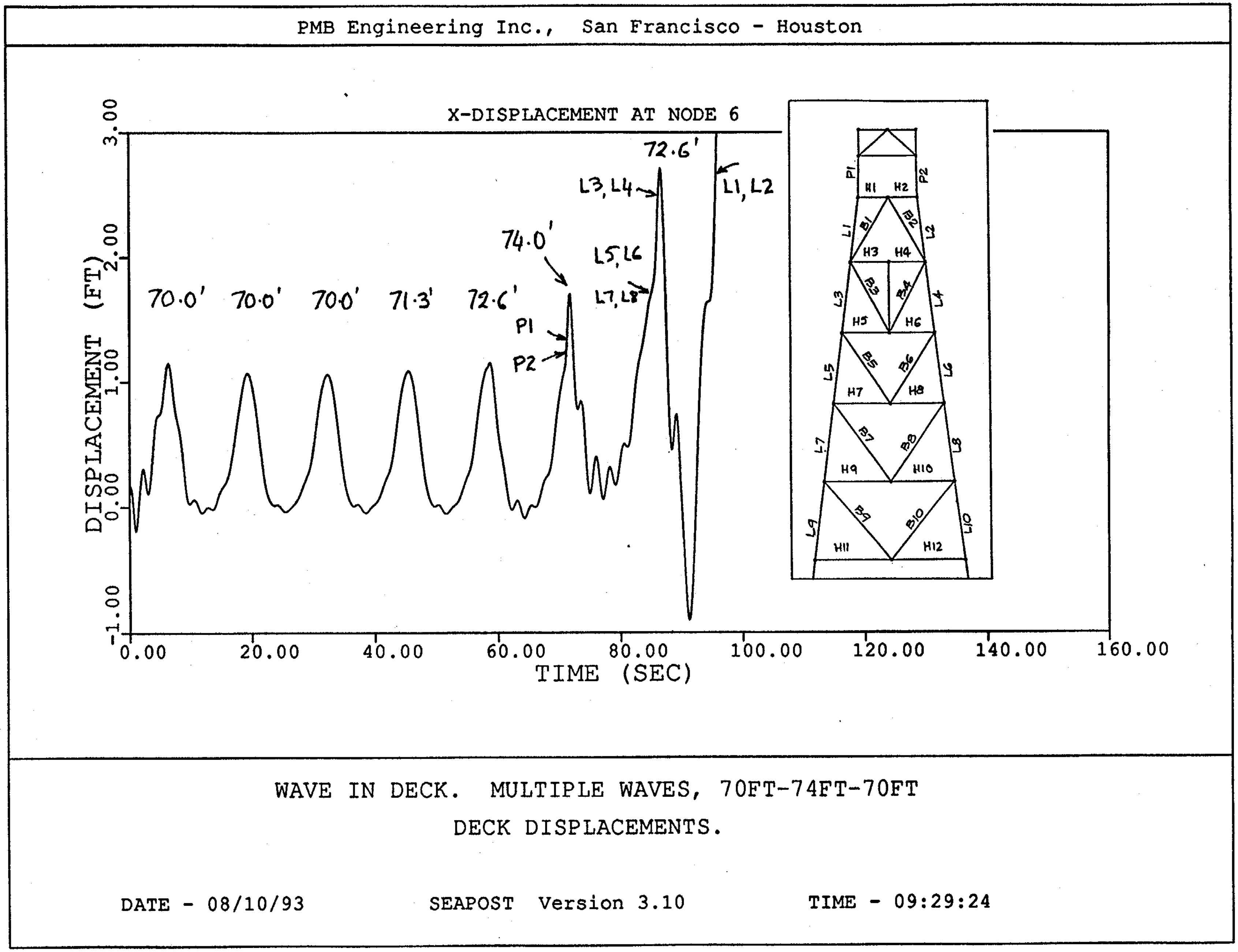
Platform C

Figure ES-1



STATIC PUSHOVER RESULTS

Fig. ES - 2



PMB Engineering Inc., San Francisco - Houston
X-DISPLACEMENT AT NODE 6
DISPLACEMENT (FT)
3.00
2.00
1.00
0.00
-1.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
TIME (SEC)
70.0'
70.0'
70.0'
71.3'
72.6'
74.0'
72.6'
P1
P2
L3, L4
L5, L6
L7, L8
L1, L2
P1
P2
H1
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L1
B1
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H7
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B7
B8
L8
H9
H10
L9
B9
B10
L10
H11
H12
WAVE IN DECK. MULTIPLE WAVES, 70FT-74FT-70FT
DECK DISPLACEMENTS.
DATE - 08/10/93
SEAPOST Version 3.10
TIME - 09:29:24

Figure ES-3

CASE DESCRIPTION	WAVE HEIGHT (ft)		APPLIED LOADS (kips)		
	STATIC PUSHOVER	DYNAMIC ANALYSIS	STATIC PUSHOVER	DYNAMIC ANALYSIS	Ratio DYNAMIC/ STATIC
Base Cases, Regular Waves :					
3-D :					
No Deck Load	80.0	78.0	4900	4700	0.96
With SHELL Deck Load	73.0		4900		
2-D :					
No Deck Load	80.5	79.0	1390	1350	0.97
With API Deck Load	74.5	75.0	1400	1430	1.02

Summary of Base Case Results
Table ES-1

CASE DESCRIPTION	WAVE HEIGHT (ft)		APPLIED LOADS (kips)		
	STATIC PUSHOVER	DYNAMIC ANALYSIS	STATIC PUSHOVER	DYNAMIC ANALYSIS	Ratio DYNAMIC/ STATIC
Base Cases, Regular Waves :					
2-D With Deck Load	74.5	75.0	1400	1430	1.02
Modified Mass and Weight :					
Jacket Mass x 5	74.0	75.0	1370	1430	1.04
Deck Mass x 2	74.0	77.0	1370	1540	1.12
Deck Mass x 3	73.0	76.0	1330	1490	1.12
Modified Stiffness and Strength :					
All Leg Thicknesses x 0.75	72.5	75.0	1300	1430	1.10
Portal Leg Thicknesses x 0.75	72.5	76.0	1300	1460	1.12

Summary of Mass and Weight Modification Results
Table ES-2

CASE DESCRIPTION	WAVE HEIGHT (ft)		APPLIED LOADS (kips)		
	STATIC PUSHOVER	DYNAMIC ANALYSIS	STATIC PUSHOVER	DYNAMIC ANALYSIS	Ratio DYNAMIC/ STATIC
Base Cases, Regular Waves :					
2-D With Deck Load	74.5	75.0	1400	1430	1.02
Modified Bracing Configuration :					
Diagonal Braces	74.5		1400		
Cross Braces	75.0	76.0	1390	1450	1.04
Diagonal Braces incl. Portal	72.0	73.5	1230	1300	1.06
Modified Stiffness and Strength :					
Ungrouted Legs	74.0	75.0	1370	1430	1.04

Summary of Configuration Modification Results
Table ES-3

Section 1
Introduction

1.1 BACKGROUND

Reassessment of existing offshore platforms is currently receiving considerable attention. There are two major environmental threats to these platforms: wave loadings from large storms, and seismic loadings from large earthquakes. Both of these issues have been studied in this project.

For wave loadings, the emphasis was on investigating how dynamic effects such as structural mass and multiple wave loading affect platform ultimate strength capacity. The intent was to determine if the industry standard "static pushover," which neglects these dynamic effects, provides a good representation of platform capacity. The wave loadings portion of the project is reported in this document.

For platforms in seismic regions, the emphasis was on documenting the current "state of practice" in seismic analysis techniques, such as linear and nonlinear modeling of the platform deck, jacket and foundation, response spectra analysis, time history analysis and pushover analysis and methods. The study also documented methods for assessing suitability of service of platforms in seismic regions. The seismic portion of the project is described in a separate document.

1.2 OBJECTIVES

The primary objectives of the wave loadings portion of the project were as follows:

- For a base case typical Gulf of Mexico platform, determine if the platform capacity determined by static pushover is equivalent to platform capacity determined by more rigorous dynamic time domain analysis.

- Use both single wave loading and multiple wave loading for the evaluation. Also use an actual wave record trace (e.g. Camille) for one of the multiple wave loading conditions.

- Vary the base case configuration (mass, stiffness, framing schemes) until differences are noted between the static and dynamic capacity. For the configurations where differences are noted, determine if the configurations are representative of actual in-service platforms (e.g. within the range of typical deck weights).

- Use rigorous analysis approaches such as three-dimensional modeling to eliminate concerns for "simplified evaluations" that may lead to erroneous results.

1.3 PROJECT PARTICIPANTS

The project was funded by the Minerals Management Service (MMS) plus seven operators. The participating organizations and their technical representatives are as follows:

CHEVRON	–	Mr. T. M. Hsu
EXXON	–	Mr. Hugh Banon
MMS	–	Mr. Charles Smith
MOBIL	–	Mr. David Petruska
PHILLIPS	–	Mr. Roger Thomas
SHELL	–	Mr. Kris Digre
TEXACO	–	Mr. Dave Wisch
UNOCAL	–	Mr. Jared Black

1.4 PROJECT TEAM

PMB Engineering Inc. was the prime contractor, providing all project management, analysis and reporting. Key PMB staff and their principal work tasks are as follows:

Project Manager	–	Frank Puskar
Lead Analyst	–	Dr. Richard Litton
Analyst	–	Dr. Stuart Pawsey
Seismic Document	–	Dan Dolan

Dr. C. Allin Cornell of Stanford University provided consulting throughout the project.

Section 2
Approach

The overall approach used for the project was to select a typical Gulf of Mexico (GOM) offshore platform, in particular one that may be sensitive to dynamic effects (e.g. deeper water with a higher natural period), perform static capacity analysis followed by dynamic capacity analysis, and then compare the static and dynamic capacities. As previously noted, sensitivity studies were also carried out to determine the impact of dynamics for different types of wave loading conditions (single and multiple waves) and different platform configurations (mass, bracing scheme).

The steps in the project were as follows:

1. **Literature Survey.** A literature survey was conducted to identify and review all pertinent information regarding static and dynamic platform capacity evaluations. The intent was to use existing information as much as possible as a basis for the study. This included not only previous direct comparisons of static versus dynamic platform capacity (as was being conducted for this project), but also information on computer modeling of offshore jackets (e.g. time to failure at tubular joints) that may have been useful for the project. A particular emphasis of the literature survey was to identify "new" sources of information not typically known to the offshore engineering community, such as similar work on bridges or buildings. Data sources included public domain literature, PMB in-house documentation and information supplied by participants. As noted later, there was little "new" information available that directly assisted the project.

2. **Select a Platform for Study.** A typical GOM platform was selected that might exhibit dynamic or time of loading effects due to wave loading conditions. Issues involved in the selection included capability for dynamic effects (natural period in the 2 to 3 second range), commonality of the platform to other GOM platforms, information available (drawings, soils), and availability of existing computer models (e.g. from the AIM projects [PMB, 1988]).

3. **Develop Platform Computer Models.** It was originally intended that all of the analyses would be performed using a fully coupled jacket-foundation three-dimensional computer model. Unfortunately, the analyses and interpretation of results for this model proved to be too time consuming for all of the analyses required for the project. Therefore, an equivalent two-dimensional model was developed and used for most of the analyses. This two-dimensional model was carefully calibrated for both static and dynamic effects against the three-dimensional model to ensure accuracy of the two-dimensional analyses. In addition, "benchmark" three-dimensional analyses were performed for both single and multiple waves to ensure that the two-dimensional results did indeed reflect results expected of the full three-dimensional model. In a separate exercise, wave

loads previously estimated for the platform using the API 17th edition wave load recipe [API, 1987] were compared to wave loads computed using the latest API 20th edition recipe [API, 1993]. PMB's CAP (Capacity Analysis Program) and SeaSTAR computer programs were used for all of the analyses.

4. **Static Pushover Analysis.** A static pushover analysis was performed on the base case two- and three-dimensional platform computer models to determine the static platform capacity for later comparison to the dynamic platform capacity. Both wave-below-the-deck and wave-in-the-deck loading conditions were considered. Of particular importance for later comparison was the equivalent wave height that caused failure. PMB's "pseudo static" pushover procedure (automated within CAP) was used for a majority of the analysis. The pseudo static approach estimates the platform capacity but does not provide details on the post-ultimate behavior. However, the pseudo static approach is more robust (easier to control) and less time consuming, allowing for a greater number of analyses. Several static pushovers performed on the two-dimensional model indicted that the pseudo static approach was providing an accurate estimate of platform capacity. This portion of the project also studied different methods proposed by the industry for estimating hydrodynamic forces when waves impact the deck.

5. **Dynamic Capacity — Regular Waves.** The dynamic capacity was found by a time history analysis of a regular wave passing by a full dynamic model of the structure. Both wave-below-the-deck and wave-in-the-deck loading conditions were considered. The dynamic model included all of the features of the computer model used for static pushover such as platform stiffness, gravity loads and wind loads, plus dynamic and time varying features such as mass, damping and changes in wave kinematics as the wave moves past the platform. Several separate dynamic analyses were required to determine the wave height first causing platform failure. This was accomplished by using the results of the static pushover as a reference level at which to run the first wave – typically taken as a few feet under the wave height estimated by static pushover. The wave height was then increased slowly for each analysis until the platform failed, the failure being defined as a state when the deck displacements increased beyond about four times the linear elastic values.

6. **Dynamic Capacity — Irregular Multiple Waves.** This effort investigated a series of waves that pass by the platforms. Two cases were evaluated. The first case consisted of a simulated storm history in which a sequence of three large waves that did not cause any platform members to fail was first passed by the platform to induce initial dynamic response. In a second sequence of five waves, the wave sizes first increased and then decreased, representing the build-up and decay of a large storm. A final sequence of three waves (similar in size to the first sequence)

determined if the platform could sustain additional loading following damage. Similar to the single wave analysis, the maximum wave sizes used in this type of load condition were increased until platform failure occurred. The size of the peak wave that caused failure was taken as the dynamic capacity. The second case of multiple wave loading used excerpts from actual recorded seastates during Hurricane Camille. Four excerpts were used in the analysis with each providing a different set of crest widths (short to long). The sizes of the waves were scaled until failure occurred, with the peak wave in the record defining the platform dynamic capacity.

7. **Variations of Base Case Platform Configuration.** As noted in later sections of this report, there was little difference between static and dynamic capacity for the base cases platform configuration. Therefore, as an additional effort to the original planned project (using funds from extra participants in the project), the base case configuration (mass, stiffness, framing schemes) was varied until the dynamic capacity differed from the static capacity. The evaluation consisted of only wave-in the deck type loading since this type of loading was determined to have the greatest probability of inducing dynamic effects. Once dynamic effects were observed (e.g. at three times the original deck mass) the "dynamically sensitive" platform configuration was evaluated to determine if it represented realistic in-service conditions.

8. **Comparisons of Static versus Dynamic Capacity.** The final portion of the project compared the static and dynamic capacities to determine if the static pushover provides a reasonable estimate of platform capacity based upon the many cases and loading conditions evaluated.

Section 3
Literature Survey

3.1 APPROACH

A limited literature survey was conducted to identify and review all pertinent information regarding static and dynamic platform capacity evaluations. The intent was to use existing information as much as possible as a basis for the study. The literature survey included issues associated with both the wave loading and seismic loading portions of the project.

Both manual (physically reviewing journals, conference proceedings, etc.) and electronic searches were used to identify potential data sources. Participants were also encouraged to identify any references not known to PMB and to offer results of in-house studies. PMB also reviewed its own in-house work for possible use on the project. The data sources were collected and briefly reviewed to see if they held information that might be important to the project. References deemed important were reviewed in-depth, with the key information used where appropriate in the project.

The literature survey was limited due to budgetary and time constraints which prevented an in-depth survey covering all possible sources and dates. However, it was felt that it was important to make at least some effort to identify potentially useful information and to prevent duplicating existing work.

3.2 DATA TOPICS AND SOURCES

Early in the literature review it became apparent that many of the identified references were already well known to the participants and PMB team. These sources included papers found in the annual Offshore Technology Conference, Offshore Mechanics and Arctic Engineering Conference, and the American Society of Civil Engineers Structural Journal. Many of these references were already identified prior to the literature search and were already slated for inclusion in the project.

Therefore, it was decided that a particular focus of the literature survey was to locate information outside the normal data sources for offshore engineering. For example, special studies at universities, specialty conferences and technology associated with other civil structures such as bridges, buildings, and spacecraft might hold some important information previously unknown.

Two electronic data sources – NTIS and Compendix, available to PMB at the University of California, Berkeley – were used to conduct an electronic data search of recent (1986-1992) technical articles. Table 3-1 shows the topics used for the search and the number of entries located for each topic. The search extracted 1830 entries that matched the topic (keyword). PMB sorted this group further, eliminating duplicates and discarding those of no direct interest.

Appendix A provides a summary, by topic, of many of the resulting papers. Included are the paper title, author, publication and a brief abstract. Table 3-2 shows several examples.

In addition to these public domain sources, PMB used several internal studies for the project such as "generic" findings of confidential site specific studies and results from several of PMB's previous Joint Industry Projects such as AIM (Assess, Inspect and Maintain) Phases I to IV [PMB, 1987a, 1987b, 1988, 1990] and HEDOP (Hydrodynamic Effects on Dynamics of Offshore Platforms) Phases I and II [PMB, 1987c, 1989, 1991].

Finally, several participants provided PMB with confidential results of in-house site specific evaluations or technical studies. This information was desensitized of confidential information and included in the studies where appropriate.

3.3 SIGNIFICANT FINDINGS

The literature survey concluded that there is little additional information useful for the project, other than work previously known to the participants and the PMB team, such as that by G. Stewart, T. Moan and R. G. Bea. As previously indicated, it is important to note that this was a "limited" literature survey. It was by no means exhaustive in terms of identifying all possible references for the topics that may be available. Such an effort was beyond the scope of this project. A more in-depth survey may identify information not located here.

However, several of the key references (by the above "known" researchers) were identified and incorporated in the study. G. Stewart (Shell) and others have been investigating dynamic effects, static pushovers, and cyclic wave loading with the findings published in several recent publications [Stewart, 1993]. R. G. Bea of the University of California at Berkeley has also been investigating dynamic platform capacity and has recently published some results [Bea, 1992, 1993]. These references are discussed where applicable throughout this report.

DATA COLLECTION

IDENTIFICATION OF TOPICS FOR SEARCH

TOPIC	NTIS	COMPENDEX
EARTHQUAKE ANALYSIS SOP:	87	124
Earthquake analysis		
Seismic analysis		
Time history analysis		
Time domain analysis		
Ductility analysis		
Coupled analysis		
DLE analysis		
SLE analysis		
Spectral analysis		
Modal analysis		
Modal combination methods		
CQC method		
Hysteresis response		
Ramp acceleration		
Deck floor response spectra		
Sources of damping		
Equivalent damping		
EARTHQUAKE RECORDS	1	0
Natural ground motion records		
Synthetic ground motion records		
Ground motion propagation		
WAVE DYNAMICS	79	204
Wave dynamic		
Multiple waves		
Wave impact		
Wave in deck		
Wave kinematic		
Wave load		
Wave force		
Low cycle fatigue		
Subtotal	**167**	**328**

Table 3 - 1

DATA COLLECTION

IDENTIFICATION OF TOPICS FOR SEARCH

TOPIC	NTIS	COMPENDEX
ULTIMATE STRENGTH OF STRUCTURES	59	95
Ultimate strength analysis		
Ultimate capacity analysis		
Collapse analysis		
Failure analysis		
Nonlinear analysis		
Cyclic analysis		
BRACE AND JOINT BEHAVIOR	13	37
Brace vibration		
Brace buckling		
Brace inelastic response		
Local vibration modes		
Brace grout		
Joint analysis		
Joint strength		
Joint inelastic response		
Joint post yield response		
Joint rupture		
SOIL STRUCTURE INTERACTION	23	63
Cyclic degradation of soil		
Soil pile interaction		
Soil structure interaction		
Strain rate effects		
Free field		
Near field		
Soil column analysis		
TESTS	50	49
Model tests		
Prototype test		
Actual vs nominal yield		
TOTAL	**312**	**572**

Table 3 - 1

(Continued)

DATA COLLECTION : SAMPLE ABSTRACTS

BRACE AND JOINT BEHAVIOR

Title: Damping in Structural Joints
Author(s): Beards, C. F.
Performing Organization: Imperial Coll. of Science and Technology, London (England).
Sponsoring Organization: National Aeronautics and Space Administration, Washington, DC.
Notes: In Vibration Inst., the Shock and Vibration Digest, Volume 21, No. 4 p 3-5.; Apr 89
Abstract: Friction damping in joints is the major source of inherent damping in most fabricated structures. Although analysis techniques are becoming more refined, it can still be difficult to accurately predict the effect of controlled joint damping on the vibration response of a structure. The range and scope of applications in which it is desirable to provide increased joint damping continues to expand.

Title: Effect of joint flexibility on seismic response parameters of steel jackets.
Author: Einashai, A. S.; Gho, W.
Corporate Source: Imperial Coll, London, Engl
Conference Title: Proceedings of the Second International Offshore and Polar Engineering Conference; San Francisco, CA, USA Conference Date: 1992 Jun 14-19
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 475-480
Abstract: In this paper, the effect of joint flexibility on the collapse mechanism of a typical deep water steel jacket structure is investigated. This is followed by an assessment of the same effect on the dynamic characteristics of the platform, in terms of periods of vibration, mode identification and spacing and modal mass distribution. Finally, the response under earthquake loading is studied, and an estimate of the structural reserve capacity (representative of the ductility and energy absorption capacity) with and without joint flexibility is obtained. It is concluded that joint compliance plays an important part in spectral force calculation, but has no effect on the modal mass distribution. The reserve strength, which is related to the seismic behaviour factor, is over-estimated by using rigid joint analysis results. (Author abstract)

Table 3 - 2

ULTIMATE STRENGTH OF STRUCTURES

Title: Reassessment of Structures: Report 0.1: Cyclic Analyses of 2D Jacket Structure
Author(s): Hellan, O.
Performing Organization: Selskapet for Industriell og Teknisk Forskning, Trondheim (Norway). Div. of Structural Engineering.
Report No: STF71-A90017; ISBN-82-595-5663-4
Abstract: The report describes cyclic analyses of a 2D jacket structure. The objective of the study has been to illustrate a methodology where nonlinear shakedown analyses are used to assess the strength of structures under variable, repeated loading. The methodology is based on FEM formulations for analysis of nonlinear structural response, extended with aspects of shakedown theory and formulations for cyclic plasticity of structural components. For a specified repeated loading, these analyses indicate whether the structure will reach shakedown (responds elastically after a number of elastoplastic cycles), or if the structure is likely to fail due to incremental collapse or reversed plasticity (when the number of cycles approach infinity). The present study shows that cyclic analyses can be used to document elastic structural response (after a number of cycles), even if the structure is loaded beyond static yielding. Under the current design practice, a situation where initial yielding occurs below the design load would require immediate and extensive actions from the operator. Now, cyclic analyses indicate that the structure can be utilized beyond the conventional ULS design limit, and still comply to the regulations' requirements.

Title: Collapse mode of elastic-plastic structures.
Author: Giambanco, F.; Panzeca, T.; Zito, M.
Corporate Source: Univ di Palermo, Palermo, Italy
Source: Journal of Engineering Mechanics v 118 n 6 Jun 1992 p 1083-1092
Abstract: For a structure of elastic-perfectly plastic material subjected to steady and cyclic loads exceeding shakedown limit, the possibility to predict collapse mode, without making a complete analysis, is illustrated. This goal is achieved by using the kinematical part of the solution to the shakedown load factor problem, and by considering that it is proportional to the gradient of the elastic-plastic, steady-state response to cyclic loads at the shakedown limit. A bounding technique, which allows the approximate assessment of any desired measure of plastic deformation occurring in the steady-state phase, is presented. Such technique differs from the usual bounding techniques because the preventive determination of only one bound on a suitable proportionality factor is requested. One the grounds of such bound value, it is possible to compute (with a very small computational effort) other bounds on any chosen measure of plastic deformation, by using the solution of the shakedown load factor problem. (Author abstract) 12 Refs.

Title: Static and dynamic analysis of collapse behaviour of steel structures.
Author: Wada, Akira; Kubota, Hideyuki
Corporate Source: Tokyo Inst of Technology, Yokohama, Jpn
Conference Title: Second World Congress on Computational Mechanics - WCCM II
Conference Location: Stuttgart, Ger Conference Date: 1990 Aug 27-31
Source: Computer Methods in Applied Mechanics and Engineering v 91 n 1-3 Oct 1991. p 1365-1378

Table 3 - 2
(Continued)

Abstract: This paper presents the computer program to pursue nonlinear behavior of steel structures until collapse state statically and dynamically and it shows two results of example analysis. The program is specially designed to use the vector-processing function of supercomputer quite effectively. ETA10 and CRAY-2 supercomputers areused in the examples. 7 Refs.

Title: Incremental collapse of structures with constant plus cyclically varying loads.
Author: Guralnick, Sidney A.; Erber, Thomas; Soudan, Osama; He, Jixing
Corporate Source: Illinois Inst of Tech., Chicago, IL, USA
Source: Journal of Structural Engineering v 117 n 6 Jun 1991 p 1815-1833
Publication Year: 1991
Abstract: Energy methods previously developed for the shakedown analyses of framed structures are extended to include structures subjected to cyclically varying loads in the presence of constant, or bias, loads. This approach is based on the hypothesis that, if the total hysteresis energy absorbed by a structure during an indefinitely prolonged repetitive loading program is unbounded, then the structure must ultimately fall. This hypothesis leads to results that are entirely consistent with the classical shakedown theorems. Several illustrative examples demonstrate that the incremental collapse envelopes of framed structures subjected to cyclically varying patterns of loading can be drastically reduced when constant loads are present. These results are essentially due to inelastic interactions between the cyclic and constant loads that magnify the effects of hysteresis. In fact, the reduction in the safe loading ranges (as defined by the incremental collapse envelope) can be a sensitive function of the constant, or bias, load components. (Author abstract) 28 Refs.

Title: Determination of the collapse load of plastic structures by the use of an upper bounding algorithm.
Author: Avdelas, A. V.
Corporate Source: Aristotle Univ, Thessaloniki, Greece
Source: Computers and Structures v 40 n 4 1991 p 1003-1008
Abstract: An upper bounding algorithm is applied to the problem of the elastoplastic analysis of structures expressed as linear complementarity problems. By the use of this algorithm, the time consuming procedure of solving large quadratic optimization problems can be avoided. Applications close the paper. (Author abstract) 25 Refs.

Table 3 - 2
(Continued)

EARTHQUAKE ANALYSIS STATE OF PRACTICE

Title: Structural model correlation using large admissible perturbations in cognate space.
Author: Bernitsas, Michael M.; Tawekal, Ricky L.
Corporate Source: Univ of Michigan, Ann Arbor, MI, USA
Source: AIAA Journal v 29 n 12 Dec 1991 p 2222-2232
Abstract: A nonlinear perturbation method is developed to solve the problem of correlating a finite element model (FEM) to a struture for which an incomplete set of natural frequencies and mode shapes and/or some static deflections have been measured. The solution algorithm can handle differences between FEM and struture, in design variables and response, as large as 100-300%, depending on the scale of the structure and correlation measures. This is achieved incrementally by making inadmissible predictions, identifying the modal cognate space relevant to the correlation measures, and making admissible corrections in the cognate space. The developed computer code postprocesses results of the FEM modal and/or static analyses of the initial model only. No additional finite element analysis is required. Lagrange multipliers reveal the dominant correlation requirements and the active admissible cognate subspace. Depending on the number of correlation variables and measures, an optimal, a unique, or an inadmissible minimal error solution may be produced. Beam and offshore tower examples are used to test the algorithm and investigate conflicting requirements, definition of admissible cognate space, limits of allowable differences between FEM and structure, accuracy, and cost of the nonlinear perturbation method. (Author abstract) 29 Refs.

Title: Jackup structures nonlinear forces and dynamic response.
Author: Winterstein, Steven R.; Loseth, Robert
Corporate Source: Stanford Univ, Stanford, CA, USA
Conference Title: Proceedings of the 3rd IFIP WG 7.5 Conference on Reliability and Optimization of Structural Systems '90
Conference Location: Berkeley, CA, USA Conference Date: 1990 Mar 26-28
Source: Lecture Notes in Engineering n 61. Publ by Springer-Verlag Berlin, Dept ZSW, Berlin 33, Ger. p 350-358
Abstract: Simple analytical methods are shown for stochastic nonlinear dynamic analysis of offshore jacket and jackup structures. Base shear forces are first modelled, and then imposed on a linear 1DOF structural model to predict responses such as deck sway. The force model retains the effects of nonlinear wave kinematics and Morison drag on base shear moments, extremes, and spectral densities. Analytical models are also given for response moments and extremes. Good agreement with simulation is found for a sample North Sea jackup. The effects of variations in environmental and structural properties are also studied. (Author abstract) 12 Refs.

Table 3 - 2
(Continued)

Section 4
Platform C

4.1 DESCRIPTION

The platform selected for study was the platform known as Platform C in the joint industry project AIM III (Assessment, Inspection, Maintenance) [PMB, 1988]. It is located in the Main Pass area of the Gulf of Mexico, located offshore from Louisiana. (Figure 4-1). Water depth at the site is 271 ft, to which was added a storm surge of 5 ft for all analyses.

The platform is a self-contained drilling platform designed in 1968-1969 and installed in 1970. There are 24 oil-producing wells on the platform. Table 4-1 summarizes the key characteristics of the platform. Figures 4-2 through 4-4 show typical elevations and plan views of the platform, detailing geometry and member sizes. The jacket is an eight leg template structure with a typical leg diameter of 44 3/4 in. and 1/2 to 5/8 in. wall thickness. Major exterior diagonal and horizontal brace sizes range from 14 to 30 in. with 3/8 to 1/2 in. wall thickness. Internal members range in size from 8 5/8 in to 18 in. with a similar range of wall thickness.

The eight 42 in. piles (Figure 4-5) penetrate to a depth of 270 ft into medium sands overlying stiff clays. Pile wall thickness ranges from 7/8 in. to 1 5/8 in. The piles extend through the jacket to an elevation of +15 ft where they extend through the top of the jacket leg to provide a connection point for the deck legs. The piles are grouted to the jacket legs.

The deck uses a truss type tubular support system (Figure 4-6) to distribute loadings. The lower deck is at about elevation 46 ft and there is an interior sump deck at about elevation 35 ft but this was not included in the structural models. The deck legs transition from 42 in. at +15 ft to 36 in. at +29 ft and extend with this size up to the upper deck level at about +35 ft.

The deck on the original AIM structure was assumed to weigh about 600 tons. Topsides equipment was 5500 tons, evenly distributed to each leg. At Meeting 3 of the current Dynamics project, participants suggested that a reduced deck weight would be more typical, and so after that date the total deck and topsides weight was reduced from 6100 tons to 2500 tons. This reduced the first natural period of the platform from 3.1 sec to 2.1 sec. Table 4-2 shows the periods of the first three modes with the two different deck weights, under the headings *6100 Ton Deck* and *Full Fnd.*

It is of interest to see how the natural periods of the jacket are affected by failures of various braces. Figures 4-7a, b, c, and d define four failure cases for each direction of loading, by listing the braces that are assumed to have failed in each. The selected failed braces were based upon the platform member failure sequence from a static pushover performed in the AIM III project. Table 4-3 shows the periods of the first three modes associated with each failure case. For the broadside loading the first sway period is little affected by the first set of brace failures, but by the time all braces in compression have been lost the period has risen from 3.07 sec to 7.47 sec.

It is also interesting to see how different parts of the structure contribute to the free vibration modes. Table 4-2 shows the periods of the first three modes, for:

1. The heavy deck, under column headed 6100 Ton Deck

2. Light deck, under column headed Full Fnd

3. With the legs fully restrained at the mudline, headed Fixed Base

4. With the whole jacket fixed, and only the portal frame and deck allowed to move

It can be seen that flexibility of the deck and foundations contribute significantly to the overall modal response.

4.2 OCEANOGRAPHIC CONDITIONS

The environmental loading on the structure comes from waves, wind and current. Unless noted otherwise, the environmental loading was from the 20th edition of API RP2A [API, 1993]. The following description and Table 4-4 summarize these environmental loadings. Table 4-2 also indicates environmental conditions from the 17th Edition of API RP2A [API, 1987] used to previously evaluate platforms in the AIM III report.

Both regular and irregular waves were used in different studies. Regular waves had a period of 13.0 sec. Synthetic irregular waves had the same period, and periods from measured storms varied wave by wave.

A surge of 5.0 ft was assumed to be present at all times.

The current was assumed to be in the direction of the waves, and was of constant magnitude 3.55 ft/sec from the surface down to a depth of 200 ft, from which point it reduced to 0.355 ft/sec at the mudline.

The wind was assumed to have a reference velocity at 10m of 80 kts, or 86 kts at the deck.

4.3 ENVIRONMENTAL FORCES

Wave forces for regular waves were computed from water particle kinematics derived from 9th order stream function wave theory including the current. A factor of 0.88 was used for the wave kinematics reduction factor specified by RP2A (20th Edition) [API, 1993].

The current velocity was scaled by a blockage factor of 0.80 resulting in a velocity of 2.84 ft/sec in the constant speed region of the water column. The mean water level current was assumed to extend from the mean water level up to the free surface, when a wave crest was passing by.

Hydrodynamic forces on the conductors were reduced by multiplying the drag and inertia coefficients by the factor 0.97, based on the spacing to diameter ratio.

Marine growth was assumed to be present from MWL down to a depth of 150 ft, and the members were assumed smooth below that. This resulted in drag and inertia coefficients as shown in Table 4-5.

The added mass coefficient was kept constant at 1.0 over the whole jacket.

Irregular waves were modeled by computing the Fourier transform of the free surface history as a set of wave components and then superimposing water particle motions calculated from each wave component, using Wheeler stretching.

Wave forces on members were calculated from the Morison equation, ignoring effects of relative velocity of water and members.

The wind force resulting from the 86 kt wind at the deck was 216.0 kips and was applied as a constant load to the deck.

Certain items just described were modified slightly to improve the agreement of the two-dimensional model with the three-dimensional model. Those will be discussed in Section 4.5.

4.4 THREE-DIMENSIONAL MODEL

The platform was modeled using PMB's CAP and SeaSTAR programs. Figure 4-8 shows the final assembled model and indicates the type of structural elements used for each member. The axis directions are shown in the inset on this figure. The Y axis, which was the direction along which the wave forces were applied, is in the broadside direction.

A brief description of these elements is provided in the following:

- **SOILS** – PSAS (Pile Soil Analysis System) elements [xx]. These nonlinear elements reflect the axial and lateral force-deformation characteristics of soil-pile interaction. The shape and character of the curves follows the recommendations outlined in API RP 2A [API, 1987]. There are approximately 10 PSAS elements located along the piles and conductors.

- **PILES/CONDUCTORS** – Nonlinear beam elements. These elements reflect the elastic-plastic relationship for beam-columns that fail by yielding in both tension and compression (no buckling) bending and torsion. These members are likely to yield before buckling due to their heavy walls and the lateral support provided by the soils. The piles are rigidly connected to the jacket legs due to grouting. The conductors are laterally supported at the conductor guide but are free to move vertically.

- **JACKET LEG/PILE** – Nonlinear beam elements. Same modeling as piles/conductors. The grouted leg/pile section through the jacket provides a sufficiently stiff section that precludes buckling.

- **DECK LEGS** – Nonlinear beam elements. Bending at the top of jacket or the lower deck connection is the likely failure mode. This is adequately modeled by the nonlinear beam element.

- **BRACES** – Struts. The slender brace members are governed by axial loading with very little bending. They are also likely to fail in compression by buckling. Therefore, these members are most properly modeled by "strut" elements that carry only axial loads and exhibit a decay in post buckling capacity (see Figure 4-10). When one of these elements has "buckled," the load capacity reduces and some of the load that was carried by the member must be redistributed to nearby members since the buckled member is incapable of carrying significant loading.

- **DECK** – Linear Beam Elements. Since the deck was modeled primarily to capture wave loads and to distribute loads between legs, the deck elements were modeled as linear beam elements. This implies that the deck elements stay within the linear regime throughout the analysis. This assumption was checked to ensure the deck elements behaved in a linear fashion. The leg members extending through the deck were modeled as nonlinear beams.

The steel material used throughout the platform is A36 with a nominal yield stress of 36 ksi. This yield value was upgraded by 12 percent to account for the difference between nominal and expected yield strengths. The yield stress was increased by another 12 percent to account for strength increases due to strain rate effects. The actual yield stress used in the analysis was therefore taken as 45 ksi (1.12 x 1.12 x 36 ksi). The intent of this modification is to account for actual in-service steel strength rather than allowable design values.

The brace members modeled with struts were checked to ensure the braces would not punch through the legs prior to buckling. This type of failure mode can be important for older structures without joint cans or grouted legs which were designed before much was known about joint punching. For this platform, the grouted leg-pile section provides adequate strength to ensure the member buckles first (i.e. the joint is stronger than the brace).

The buckling capacity of the braces was also modified to account for lateral wave loading. This decreases the buckling capacity due to lateral loads acting along the length of the member. Figure 4-11 shows the response for a typical brace and the effect of lateral wave loading. For this platform and these wave conditions the buckling load is only slightly affected.

The conductors were modeled as three individual elements with each element representing eight conductors. This simplification is used to reduce the size of the computer model.

The interior horizontal framing was modeled in detail (except the conductor trays) to ensure the proper load paths for load redistribution once platform members fail. The typical approach of representing this framing with an equivalent "X" brace may result in improper load redistribution and hence was not used here.

Initially the kinematics factor to be applied to regular wave kinematics was 0.88 in agreement with the 20th edition of API RP2A [API, 1993]. It was found that a wave of height 69 ft caused collapse in static pushover, without including deck wave loads. Inclusion of deck loads would not have made any difference to this pushover capacity since the crest elevation of the wave was below the lower deck. It was decided to adjust the model so that a larger wave would produce static collapse. Thus the kinematics factor for regular waves was changed to 0.75. The resulting model failed in static pushover with an 80 ft wave, when deck wave loads were not considered.

Damping of 5% on the first natural mode was used for all analyses.

4.5 TWO-DIMENSIONAL MODEL

Because it would have been extremely time-consuming to perform all the studies on the full three-dimensional model, a simpler two-dimensional model was constructed and was used for most of the analyses (Figures 4-12, 4-13). This model represents one quarter of the three-dimensional model and consists of one transverse frame of the structure. The axes are shown in Figure 4-12. Note that the X axis in this two-dimensional model corresponds with the Y-axis in the three-dimensional model. All main jacket members are the same as those in the three-dimensional model but considerable simplifications were made regarding the modeling of the soil and members out of the plane of the two-dimensional model.

The pile-soil system was represented by a linear beam at each leg, the lower end of which was fixed (Figure 4-12). The stiffnesses of these beams were adjusted so that the total stiffness of the three-dimensional and two-dimensional jackets at the deck was the same (Figure 4-14).

The conductors, all structural members out of the plane of the two-dimensional frame, and all other wave-loaded members were represented by a pair of vertical beam elements at approximately the same position as the legs. (Figure 4-15). These conductor sticks had stiffness low compared with the jacket, and were constrained to move with the jacket at each elevation. The drag coefficient of these sticks was adjusted level-by-level to give the same drag force as that of the three-dimensional model, when subjected to a uniform current.

As in the three-dimensional model, the legs and piles were represented by nonlinear beam elements, the horizontal and diagonal braces by nonlinear struts, and the deck by linear beams (Figure 4-12.)

A static pushover test was then made on the two-dimensional model and this was compared with that from the three-dimensional model. It was found that the two-dimensional model failed at a somewhat higher wave height than the three-dimensional model. This was largely because, when loaded transversely, the loading on the three-dimensional model is rather asymmetric because of the large drag forces on the conductors that are not positioned in middle of the jacket. Failure of the frame at the jacket end nearest the conductors precipitated progressive failure of the whole jacket at a somewhat lower load than occurred when the same loads were applied evenly to all frames. In order that the two-dimensional model would fail at the same wave height, the drag coefficients on the conductor stick were increased uniformly by about 25%. With this adjustment, the two-dimensional and three-dimensional models both collapsed with an 80 ft wave (wave-below-deck condition), when applied statically.

Masses were matched elevation by elevation with those from the three-dimensional structure. The first natural periods are compared with those of the three-dimensional structure in Table 4-6.

It is of interest to see how the natural periods of this two-dimensional jacket model are affected by failures of various braces. Figures 4-16 and 4-17 show the first two mode shapes for the intact jacket, with the heavy deck. Figures 4-18 and 4-19 show the effect of the failure of brace B3, while Figures 4-20 and 4-21 show the effect of failures of four braces in compression from a wave load. It can be seen that one brace failure affects the mode shape considerably, but the period only changes by about 10%. However by the time four braces have failed, the first period had increased from 2.72 sec to 13.2 sec. This observation is similar to that found for the three-dimensional computer model (Section 4.1). The platform's natural period is not affected significantly until multiple members have failed.

Damping was specified as 5% of critical on the first natural mode, as in the three-dimensional structure.

Other comparisons were made between the three-dimensional and two-dimensional models and these will be described in Section 6.

4.6 WAVE FORCES ACTING ON THE PLATFORM DECK

4.6.1 Background

Wave forces acting on the deck are not typically an issue for new platform designs but are important for assessment of older existing structures. These structures were at times designed with inadequate oceanographic criteria resulting in a deck elevation that according to today's design criteria, may be impacted by large waves. This phenomenon was confirmed during Hurricane Andrew where scores of platforms were impacted by large waves that reached into the deck [PMB, 1993]. In some cases these platforms survived, and in some cases these platforms were damaged or failed.

Wave forces acting on the deck are particularly important since they can be quite large and may be as much as 20 to 40 percent of the total base shear acting on the platform at collapse. The centroid of the deck wave force is also located at a high elevation resulting in a high overturning moment. For some platforms, the deck wave force may cause local failure of the deck legs due to portal action with little effect on the jacket or foundation.

In terms of dynamic effects, the large magnitude of the deck wave force as well as the short amount of time that it acts on the structure are important. In some respects the "localized" wave deck force, typically 2 to 3 seconds in duration as the wave peak passes through the deck, can be thought of as an impulse-type loading compared to the "global" wave force acting on the combined jacket-deck system where the wave may take 10-14 seconds to pass through the platform system. For impulse type loadings, the mass of the platform system may cause the platform to react in a "slow" manner, resulting in forces less than that predicted by static loads alone.

For these reasons, it was felt important to include cases where the deck is exposed to waves. Since the primary concerns for dynamics are the magnitude of deck loadings and the time history of load application, it is important to accurately predict and model the time history deck forces. The following section describes several alternative approaches for estimating deck wave forces that were studied in the project.

4.6.2 Available Approaches

Three approaches were available for use by the project: 1) AIM simplified procedure, 2) participant in-house procedures, and 3) API task group deck force guidelines. Each is briefly described below.

1. **AIM Simplified.** The AIM projects [PMB, 1987, 1988, 1990] used a simplified procedure for estimating deck wave forces that was based upon the overall projected vertical area of the deck. The procedure was similar to computing deck wind forces except for use of wave kinematics combined with different drag coefficients based upon work by DnV [DnV, 1977]. In addition, the crest elevation and kinematics were modified to account for wave runup. Figure 4-22 shows the resulting AIM deck forces for various wave heights in the broadside direction. The AIM procedure does not provide any time of loading; however, as part of this study, an estimate of the time of loading was developed based upon the time it takes the wave crest to pass through the deck as shown in Figure 4-23a,b. The time computed based upon the lowest deck elevation where the wave hits the deck, the platform width, the wave shape (as computed by Stream Function) and the wave celerity. Figure 4-24 shows the resulting time of loading (deck inundation) per these conditions. It was assumed that the load ramps up in a sinusoidal manner with the peak load occurring at one-half of the total time that the wave crest is in the deck.

2. **Participant In-house Procedures.** Chevron and Shell offered use of their in-house deck wave force procedure for use in the project. Both procedures are based upon proprietary laboratory tests of waves impacting the decks of several scale model platforms.

The Chevron procedure relies on explicit computer modeling of the deck structure (all tubulars and wide flange beams), equipment and deck grating with the use of special drag coefficients for each. The wave of interest is then run past the platform computer model with the deck forces computed as the wave crest passes through the deck, impacting each of the explicitly modeled deck elements. Unfortunately, for Platform C, only very poor quality drawings were available of the deck framing and no information was available of deck equipment. Therefore, it was not possible to develop a detailed model of the deck for use of the Chevron procedure.

The Shell procedure uses the projected area of the deck plus proprietary drag coefficients to determine the deck wave forces. The Shell procedure also develops a time history of the deck wave forces. The Shell deck forces were based on assuming a dense deck (heavily blocked), 2 dimensional waves (no spreading), and using the envelope (upper bound) of estimated deck forces. Shell computed the deck wave forces in-house, based upon drawings of Platform C provided by PMB, for the broadside, diagonal and end-on directions. Figure 4-22a shows the Shell maximum deck forces as a function of wave height. Figure 4-25 shows the time history shell deck wave forces for the broadside direction with no current. These forces were used in various dynamic analyses.

It was later decided that less equipment would have been more appropriate and a directional spreading factor could have been used. With these adjustments, the Shell deck forces shown in Figure 4-22b resulted.

3. **API Task Group Deck Force Guidelines.** A subcommittee of the API Task Group 92-5 on Assessment of Existing Platforms for Fitness of Purpose has proposed preliminary guidelines for a simple yet conservative method for predicting deck wave forces. Table 4-7 summarizes the API approach which is also based upon the projected area of the deck. The API approach also accounts for directional spreading and contains a correction factor for the wave height in order to estimate crest elevations in closer agreement with measured data. The API procedure does not include time variation of deck forces. Figure 4-22b shows the resulting maximum API deck forces as a function of wave height for the broadside direction assuming a moderately equipped older deck (halfway between heavily and bare deck). These deck forces are seen to be quite similar to the adjusted Shell forces, which were based on the same assumptions of spreading and deck density.

4.6.3 Platform C Deck Forces

Figure 4-22b shows a comparison of loads computed by the three approaches with consistent assumptions. The AIM values are expected to be the least accurate since they are based upon simple theory and not laboratory tests as are the Shell and API methods, which agree quite well over the important range of 72 to 78 ft.

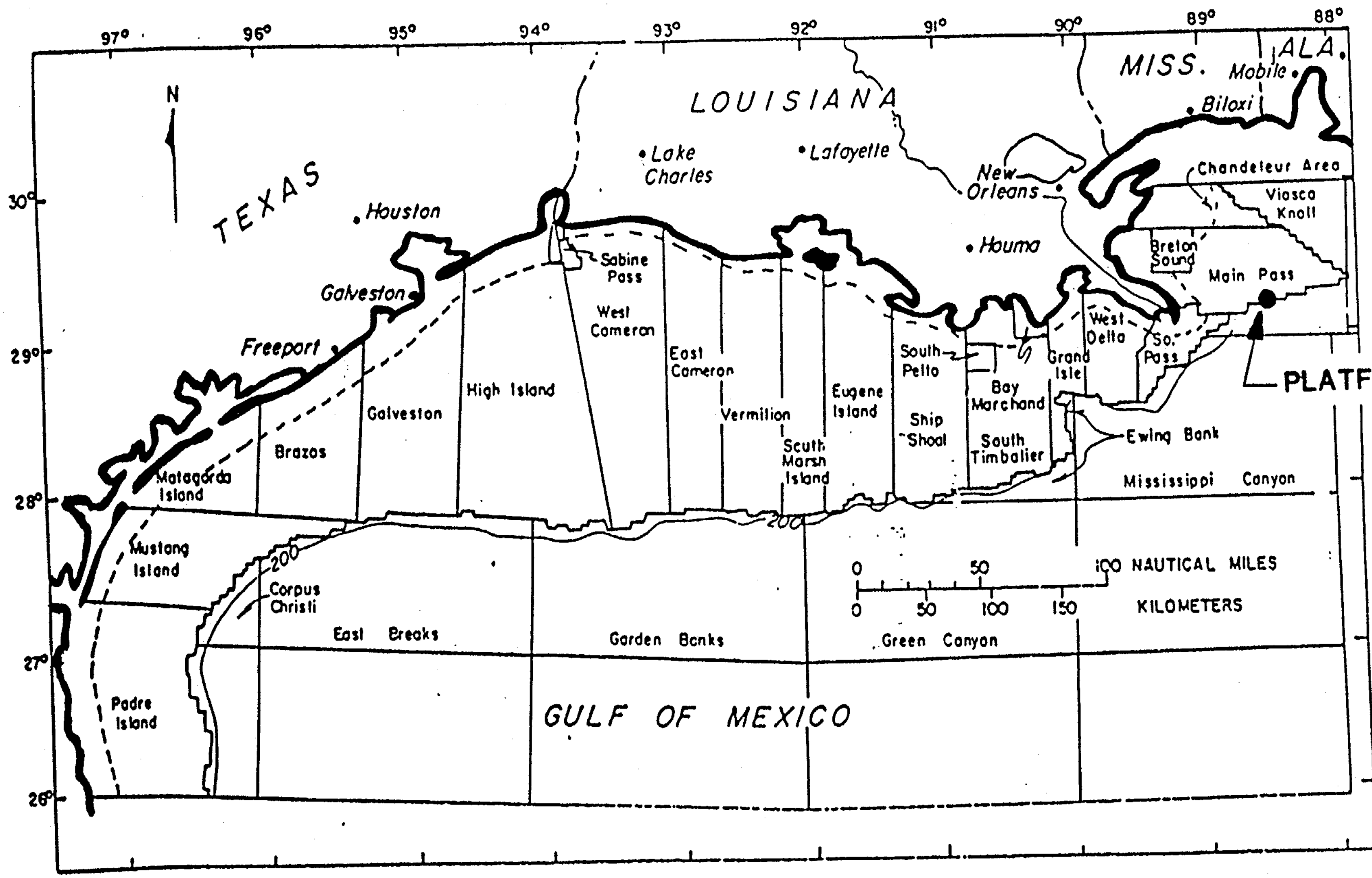
The Shell deck wave forces were used for some of the earlier analysis for the project. The deck loads were later revised to the API approach since this appears to be the most appropriate approach for Platform C. Since no time of loading information was available for the API approach, the approach used by PMB earlier in the project (see previous discussion of the AIM approach, Figure 4-24) was used to establish time of loading. This was felt acceptable since the sinusoidal shape of this approach closely matched the shape of the Shell time history deck wave forces.

For static loading, the maximum deck forces for a particular wave were applied to the deck leg nodes as a portion of the static base shear acting on the platform at time of failure. For dynamic loading, the time history deck forces were added to the time history jacket forces, resulting in a sudden increase in platform forces as the wave crest passes through the deck, as shown in Figure 4-26. Further details of how the wave deck forces were used are provided where required during discussion of a particular case study.

4.7 WAVE FORCE COMPARISON (API 17th vs. 20th Edition)

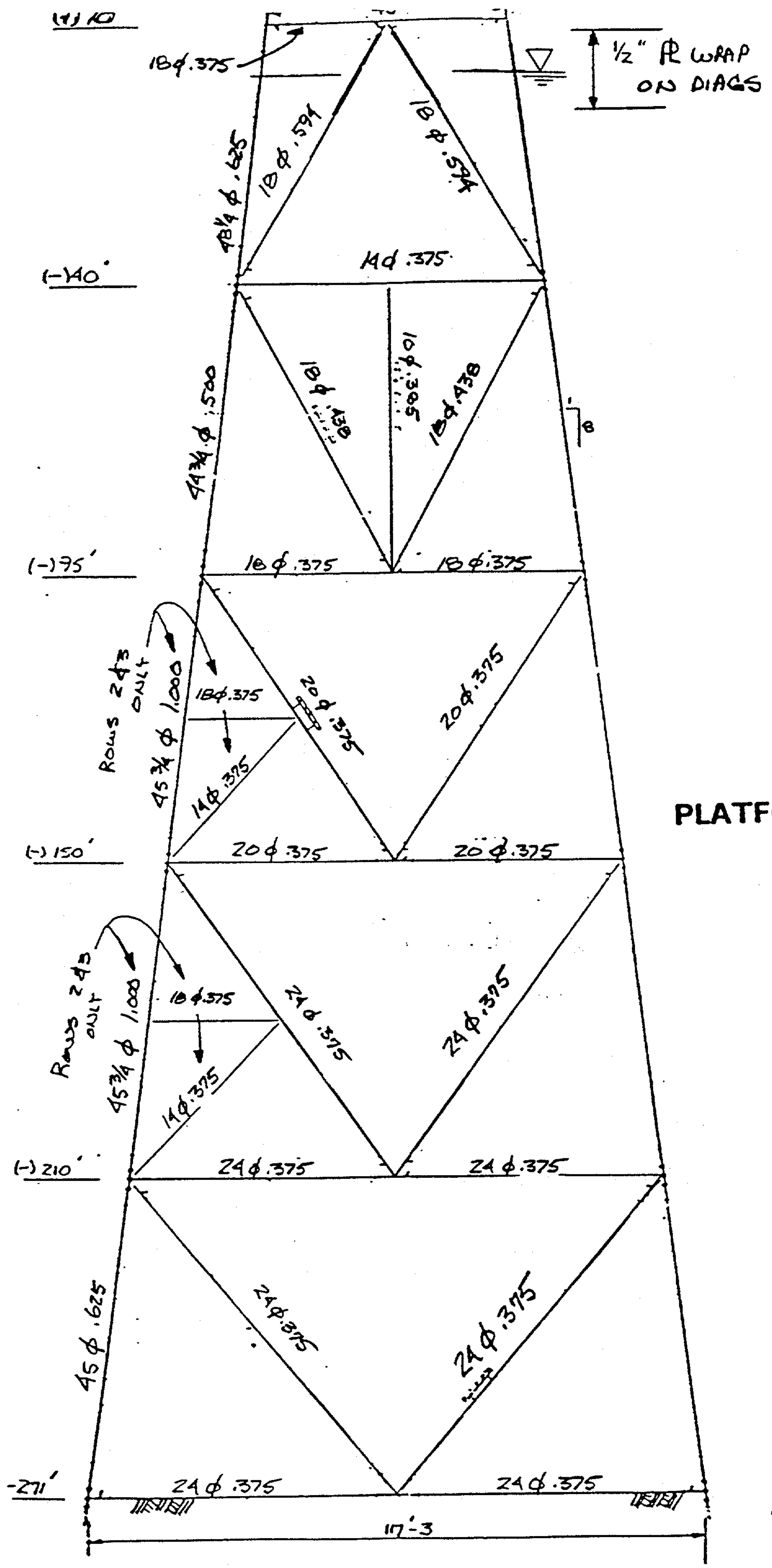
Wave forces acting on Platform C according to the 17th edition API RP 2A wave load recipe [API, 1987] and the 20th edition API RP2A wave load recipe [API, 1993] were compared. Table 4-4 shows the major differences between the two recipes. Platform directionality was not considered in the comparison (e.g. the same wave was considered from all directions).

Figure 4-27 shows a comparison of the two wave load recipes as a function of wave height. The 20th edition recipe results in wave (and current) loads that are approximately 60 to 70 percent higher than wave loads computed by the 18th edition recipe. Table 4-8 summarizes the major force contributions between the two procedures. Once directionality is considered, the differences between the two procedures would decrease.



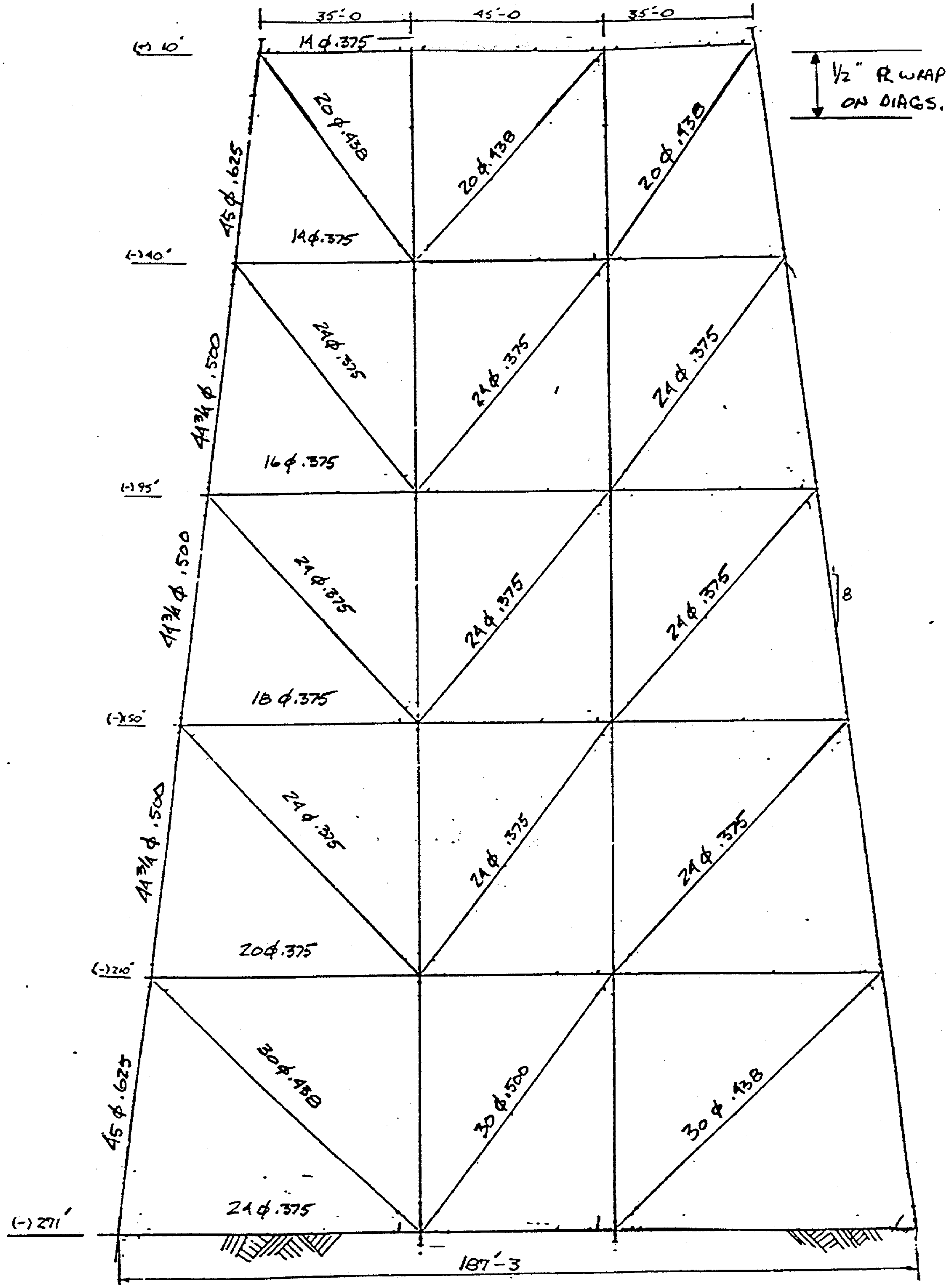
PLATFORM "C" LOCATION

Figure 4 - 1



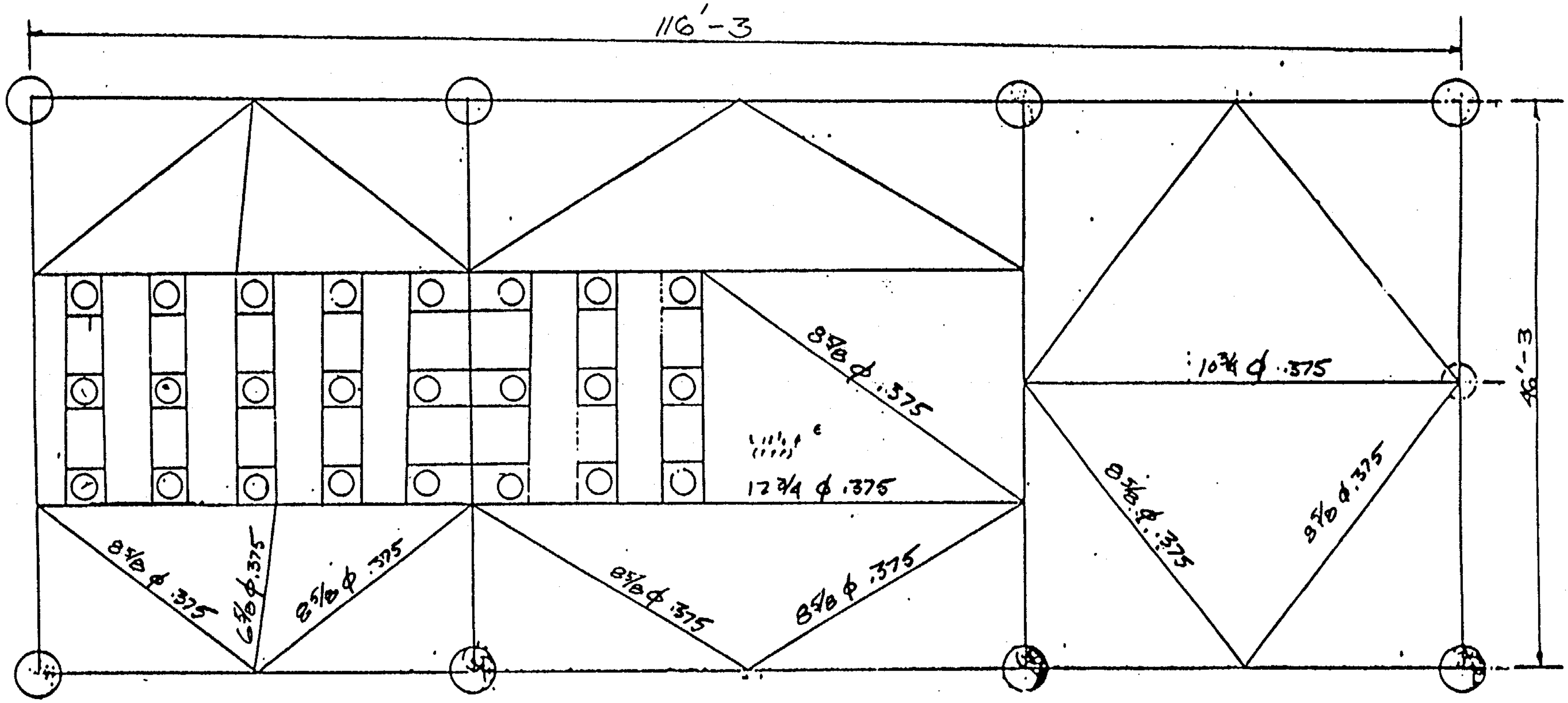
PLATFORM "C" ROWS 1 - 4

Figure 4 - 2



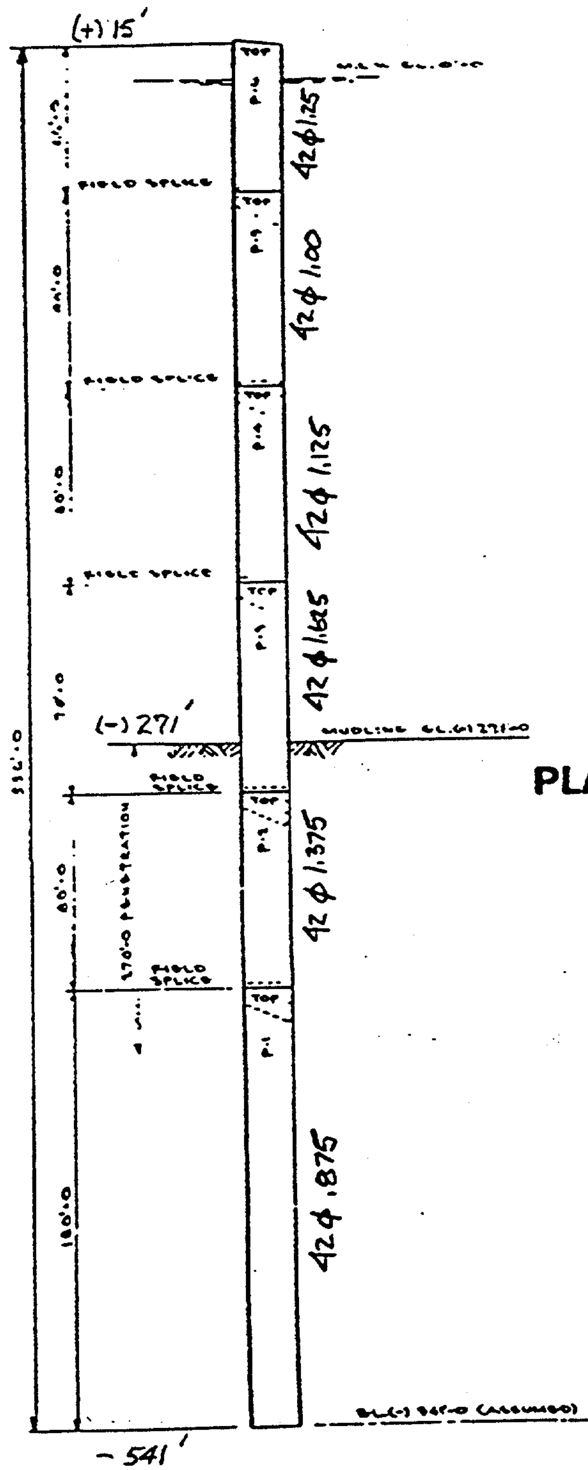
PLATFORM "C" ROWS A - B

Figure 4 - 3



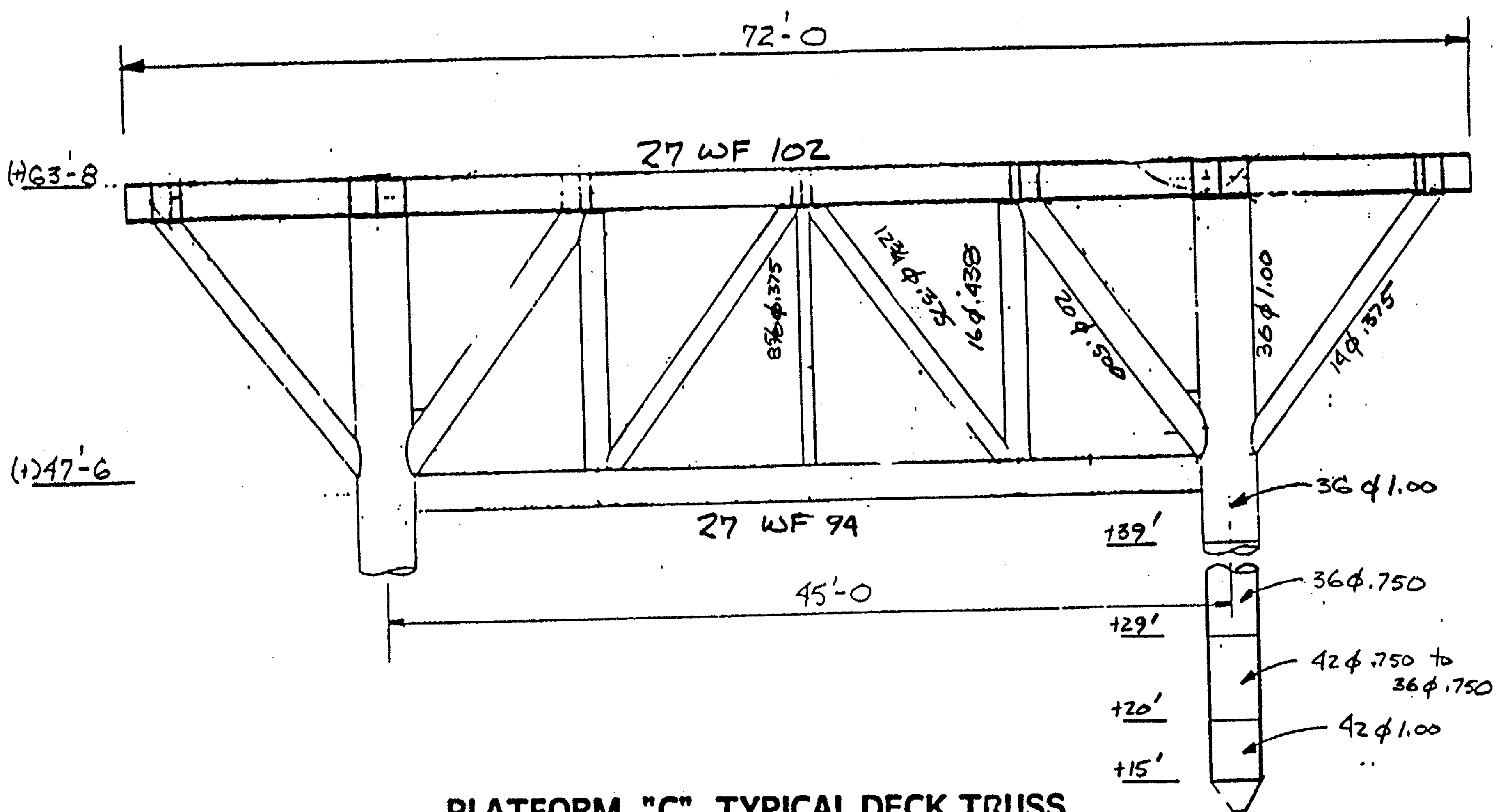
PLATFORM "C" TYPICAL PLAN VIEWS

Figure 4 - 4



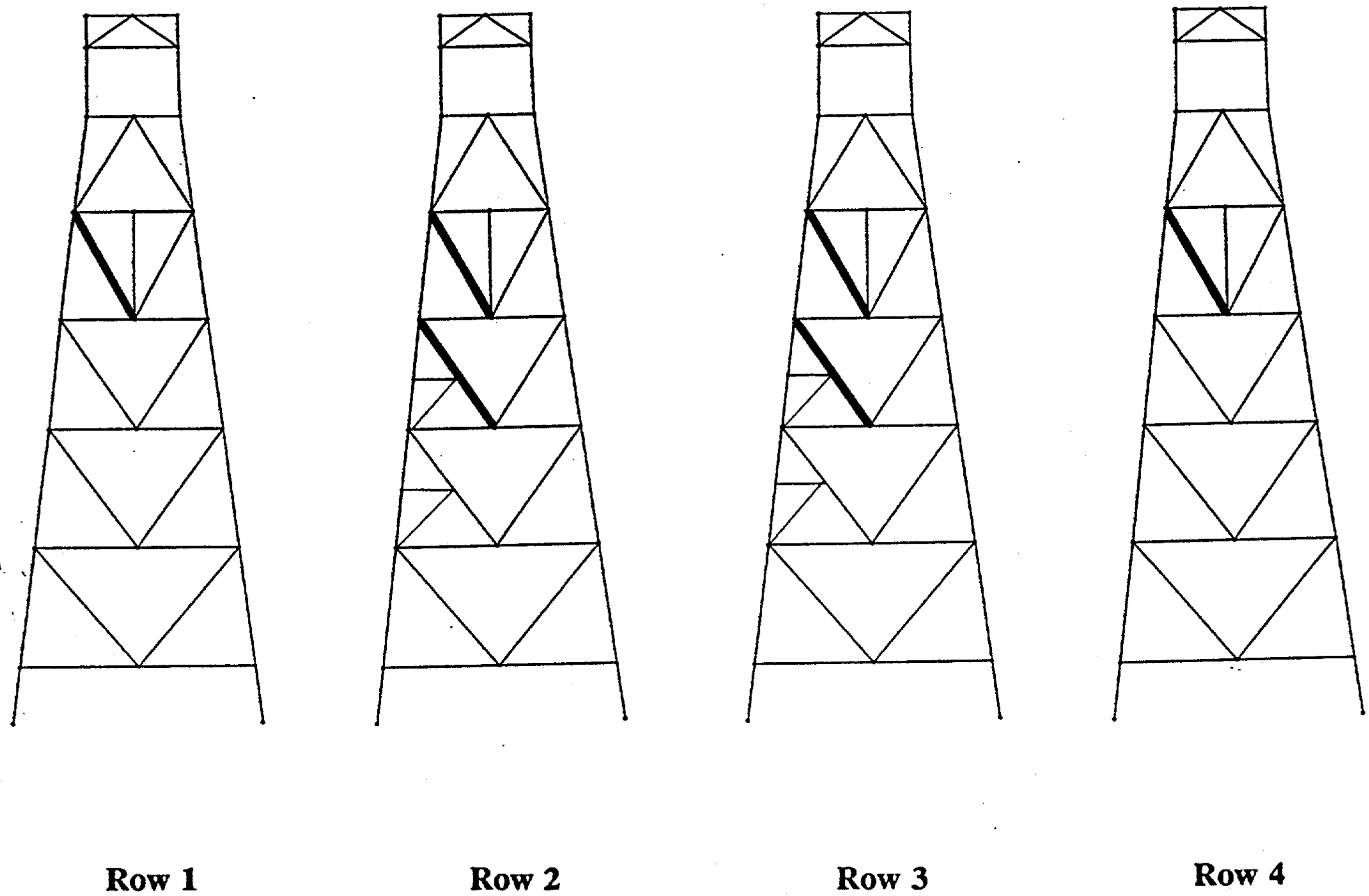
PLATFORM "C" PILE CONFIGURATION

Figure 4 - 5



PLATFORM "C" TYPICAL DECK TRUSS

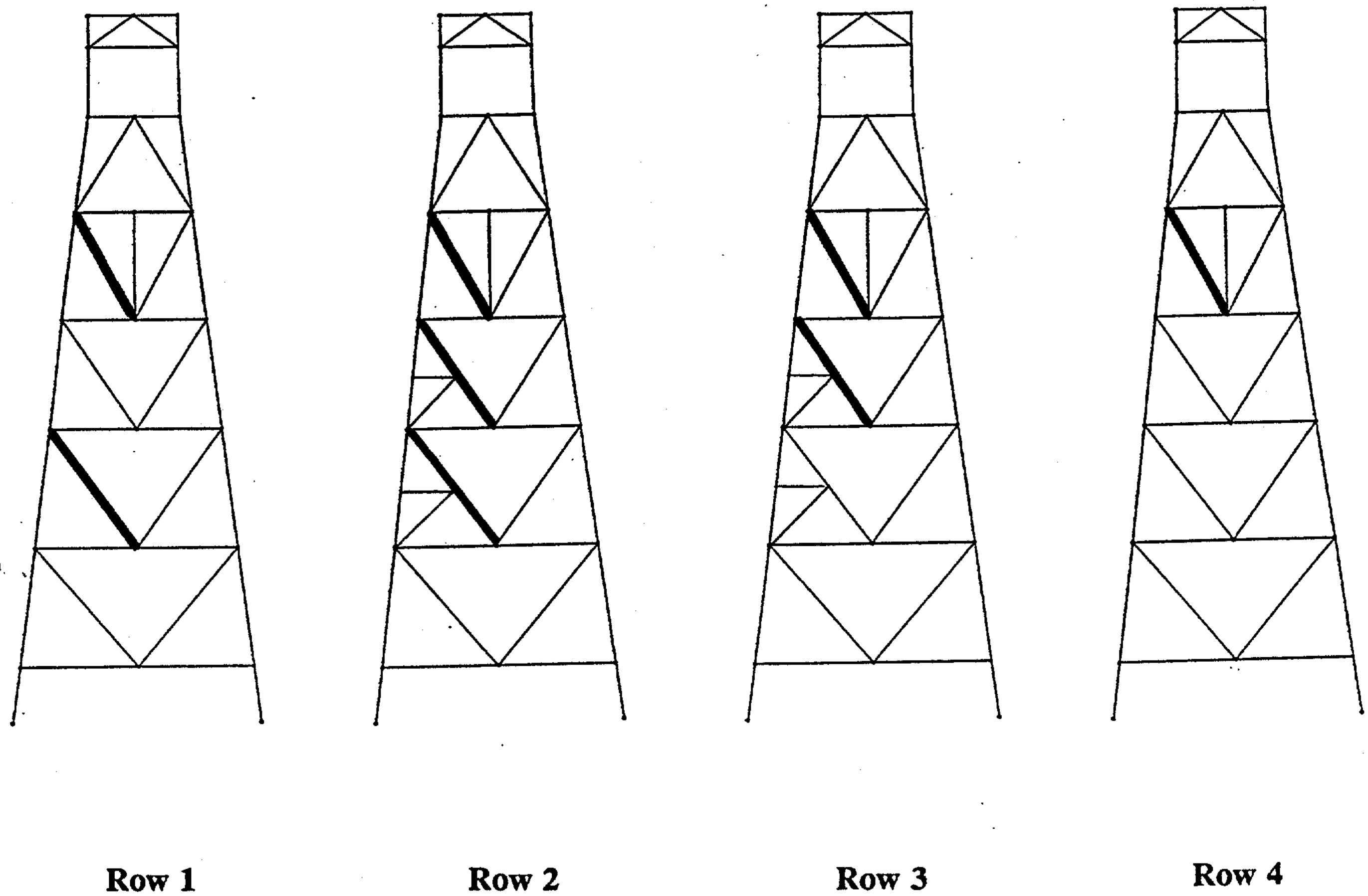
Figure 4 - 6



Broadside Load, Case 1.

Case Definitions for Frequency Analyses With Damaged Members

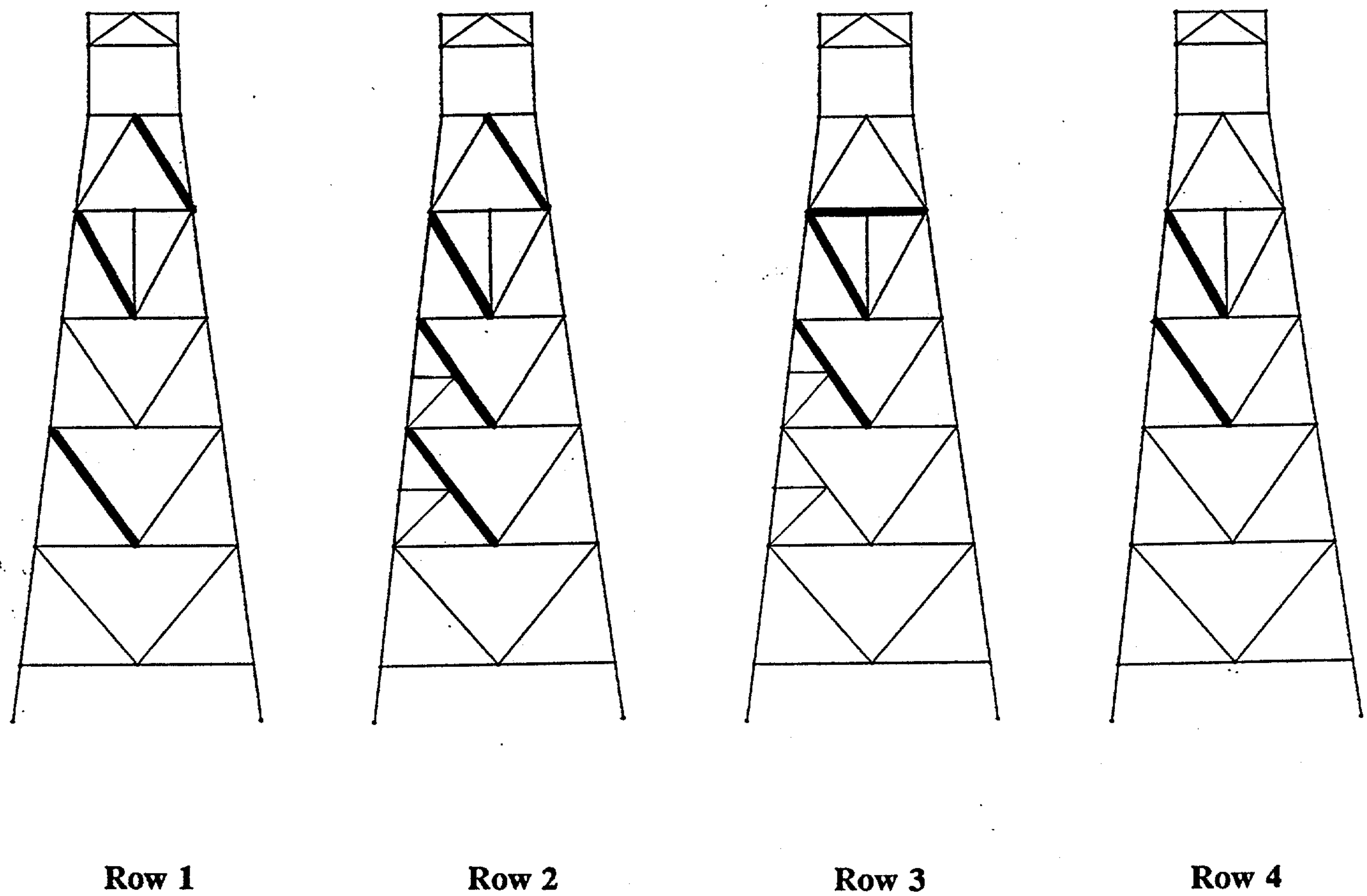
Figure 4-7a



Broadside Load, Case 2.

Case Definitions for Frequency Analyses With Damaged Members

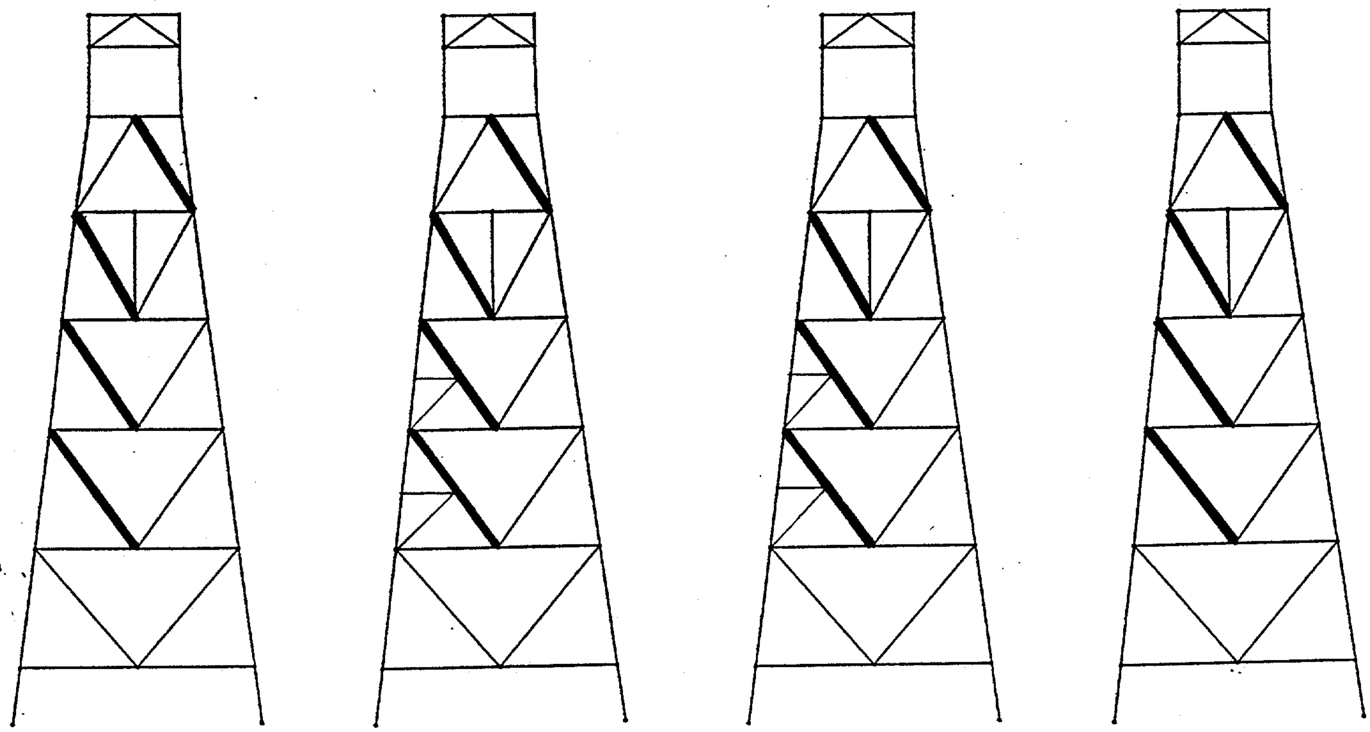
Figure 4-7b



Broadside Load, Case 3.

Case Definitions for Frequency Analyses With Damaged Members

Figure 4-7c

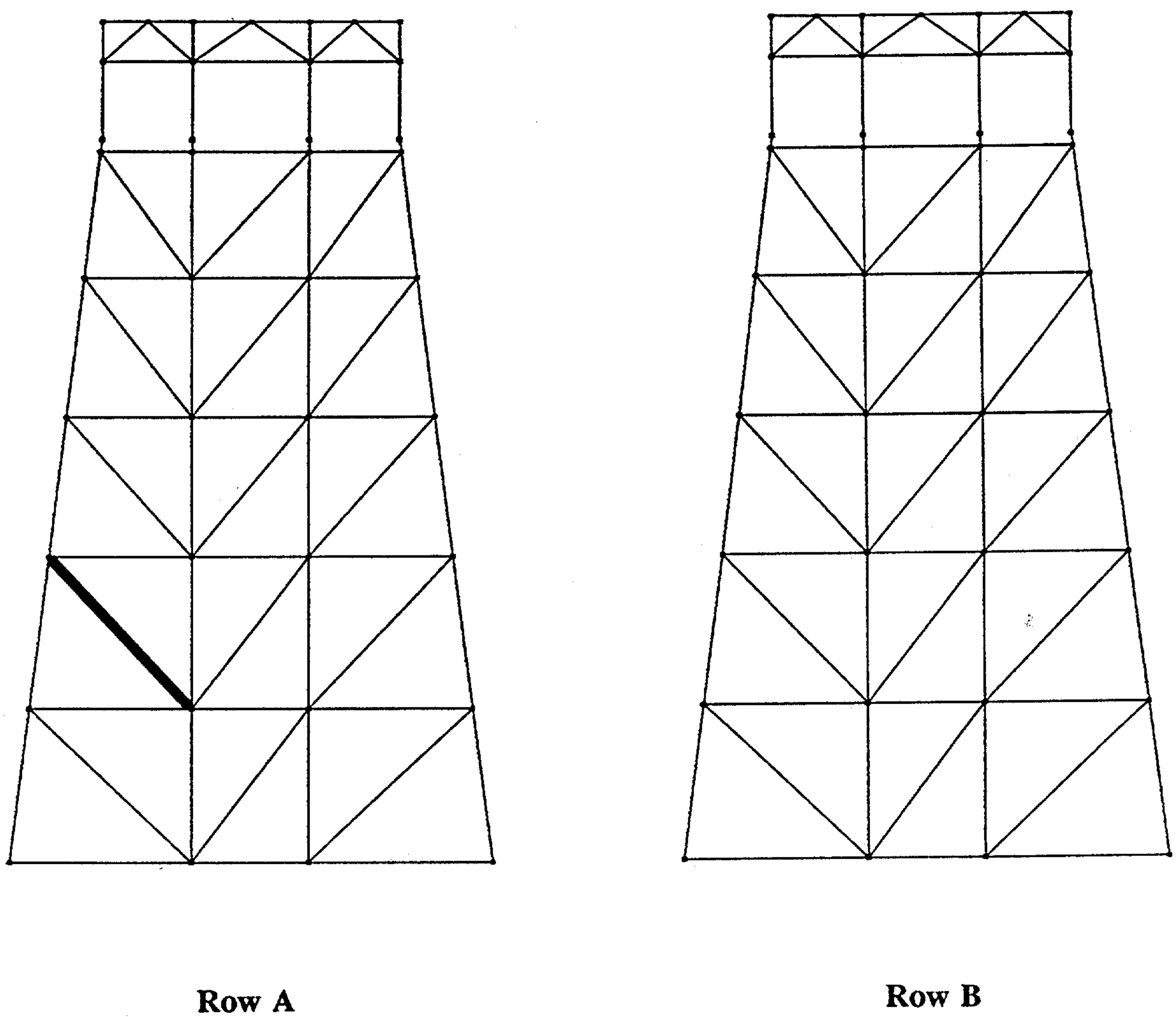


Row 1 Row 2 Row 3 Row 4

Broadside Load, Case 4.

Case Definitions for Frequency Analyses With Damaged Members

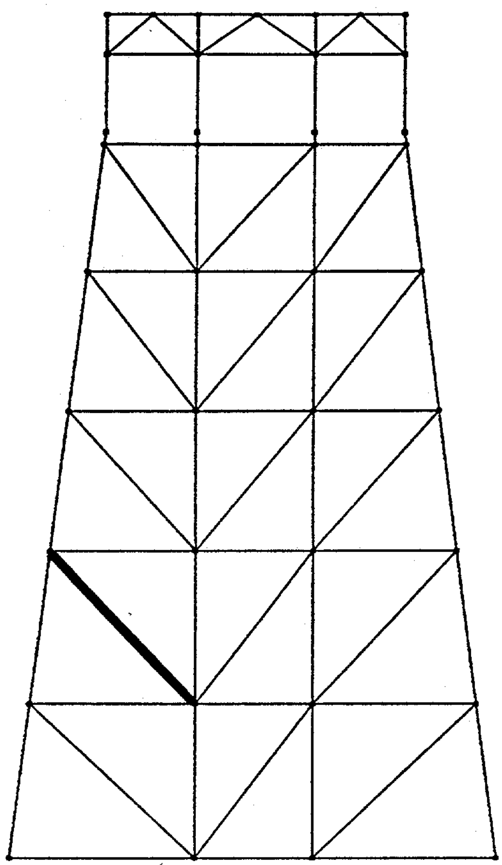
Figure 4-7d



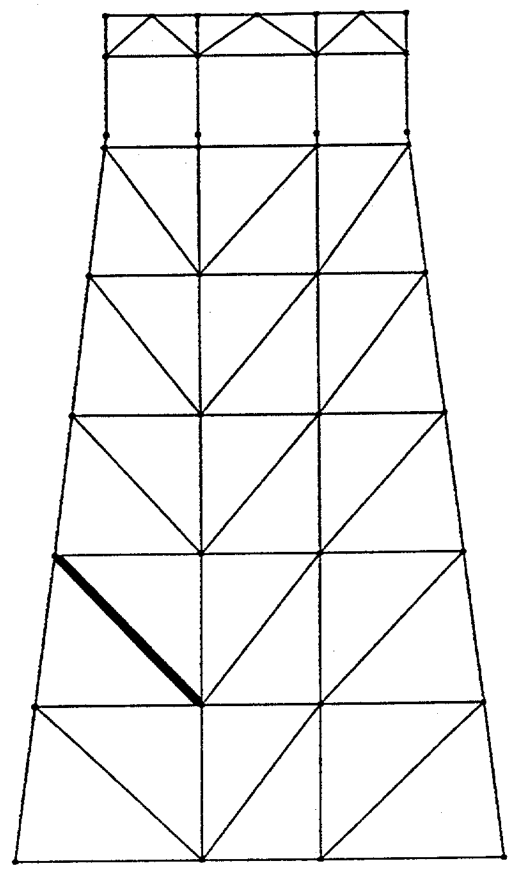
End-On Load, Case 1.

Case Definitions for Frequency Analyses With Damaged Members

Figure 4-7e



Row A

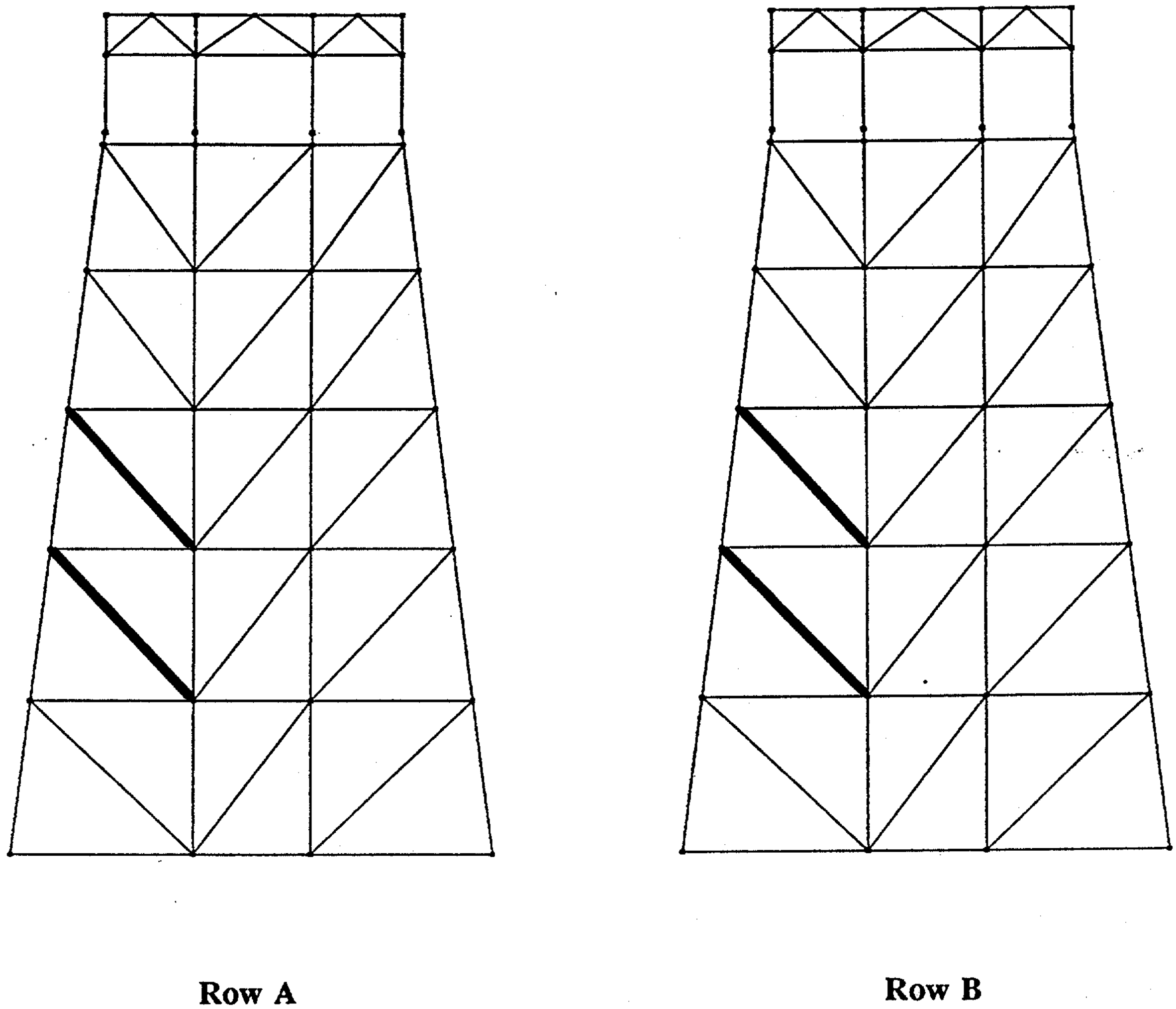


Row B

End-On Load, Case 2.

Case Definitions for Frequency Analyses With Damaged Members

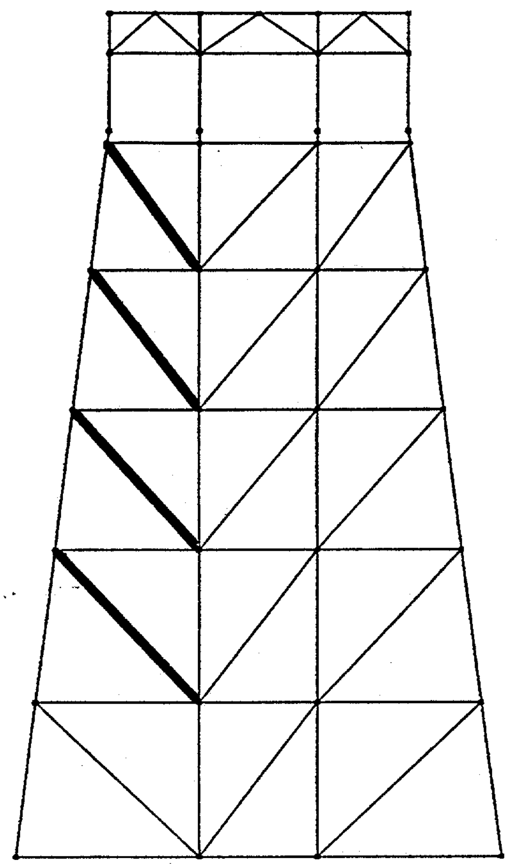
Figure 4-7f



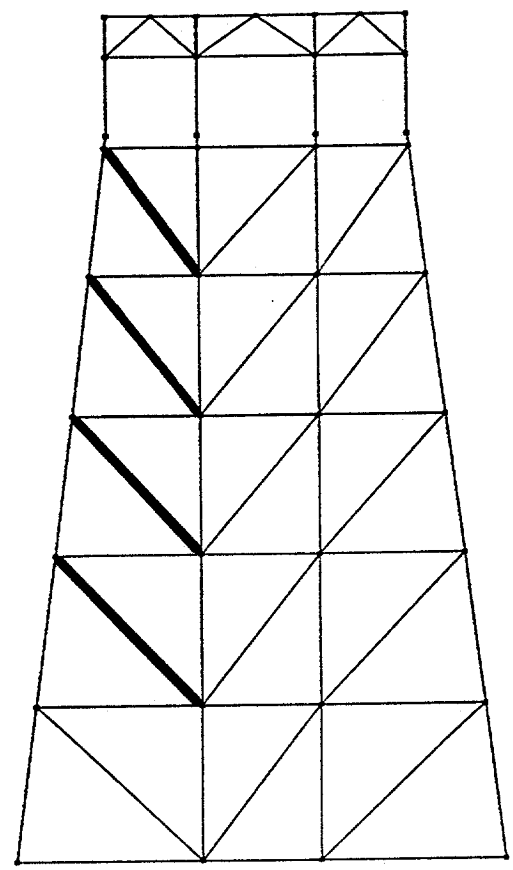
End-On Load, Case 3.

Case Definitions for Frequency Analyses With Damaged Members

Figure 4-7g



Row A

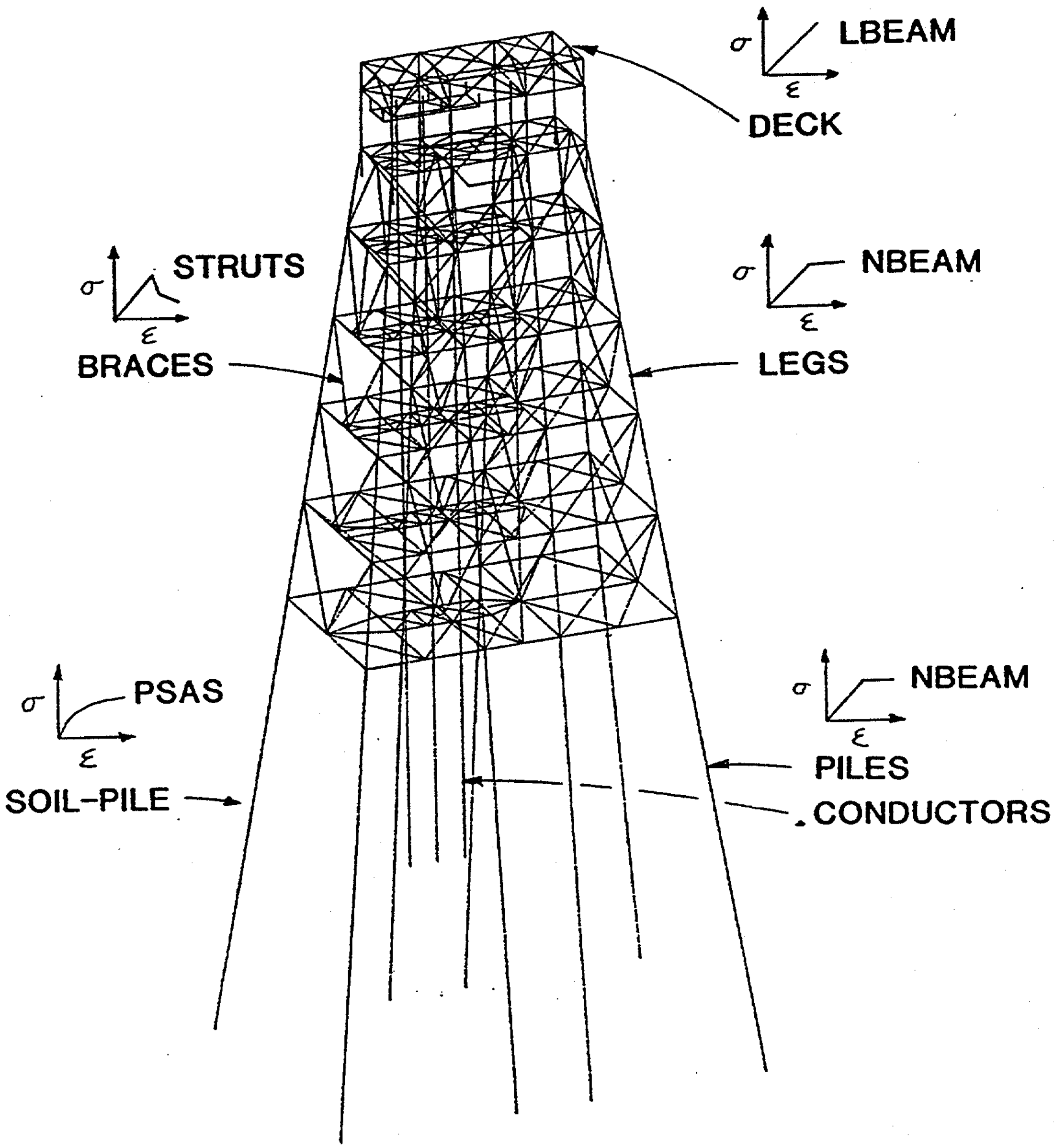


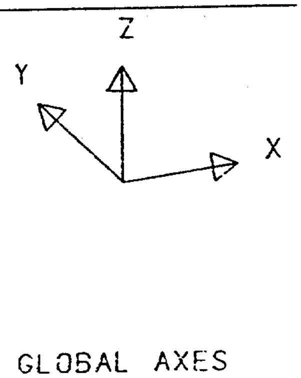
Row B

End-On Load, Case 4.

Case Definitions for Frequency Analyses With Damaged Members

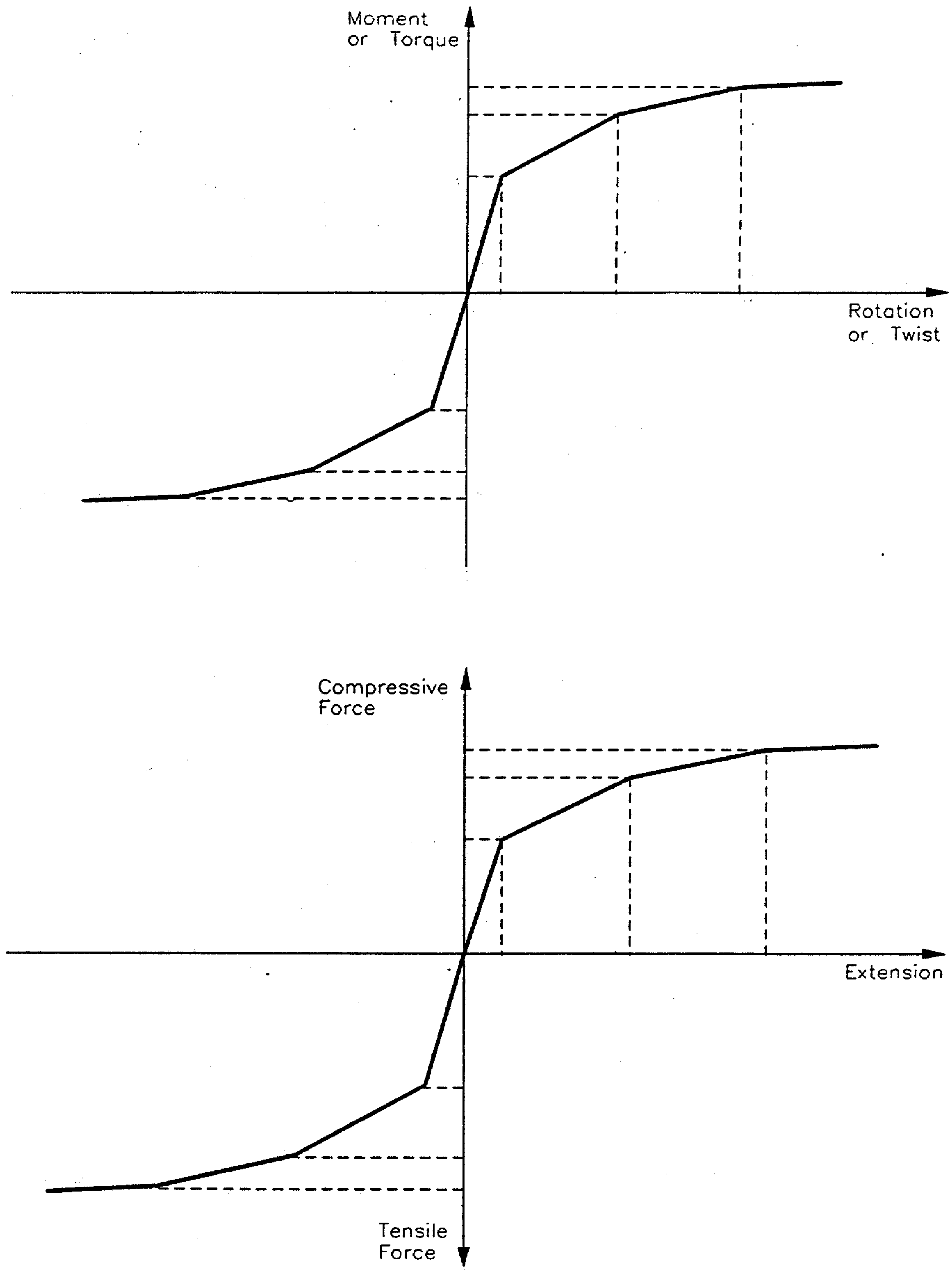
Figure 4-7h





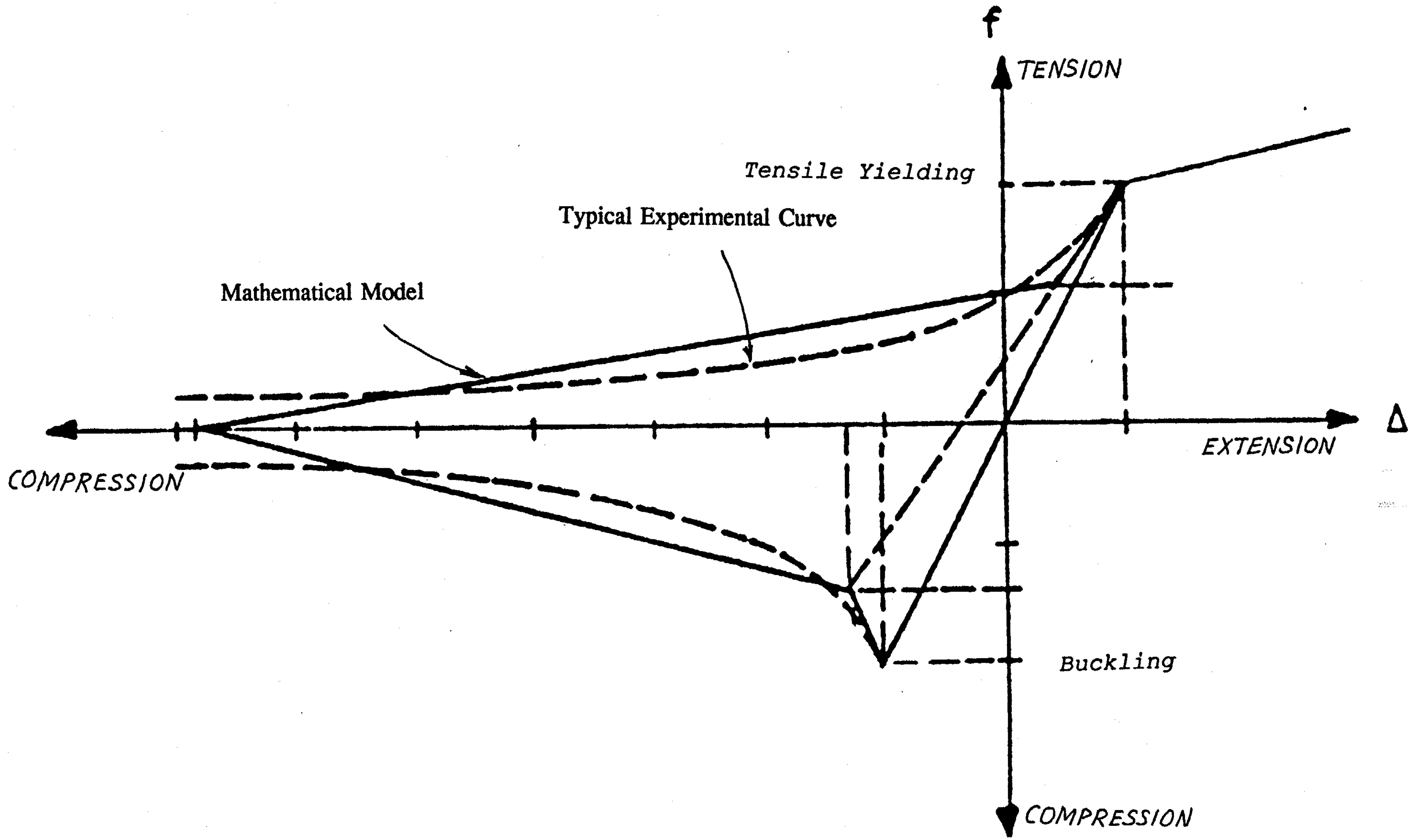
NONLINEAR COMPUTER MODEL

Figure 4 - 8



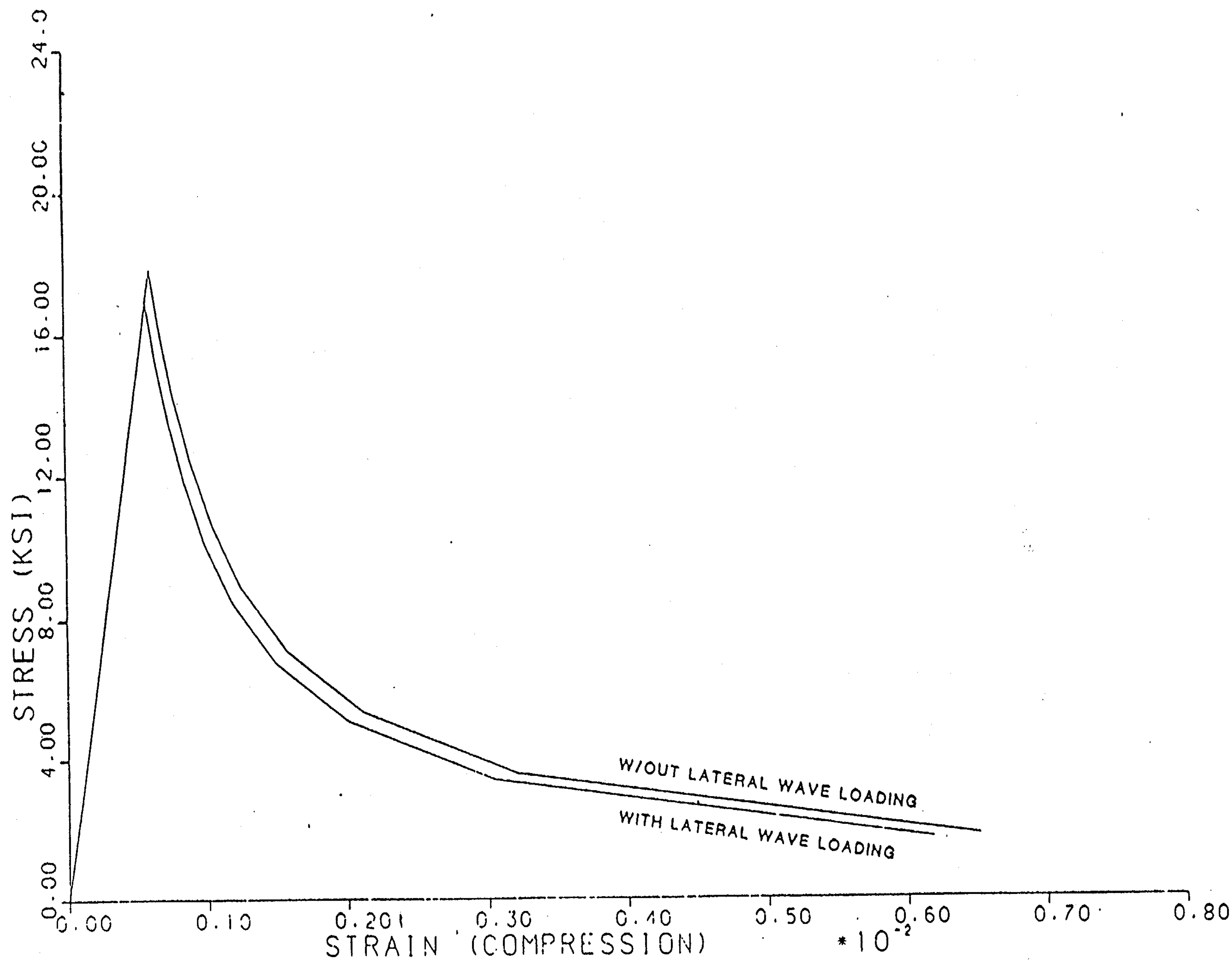
ACTION VS. DEFORMATION FOR BEAM-COLUMN ELEMENT

Figure 4 - 9



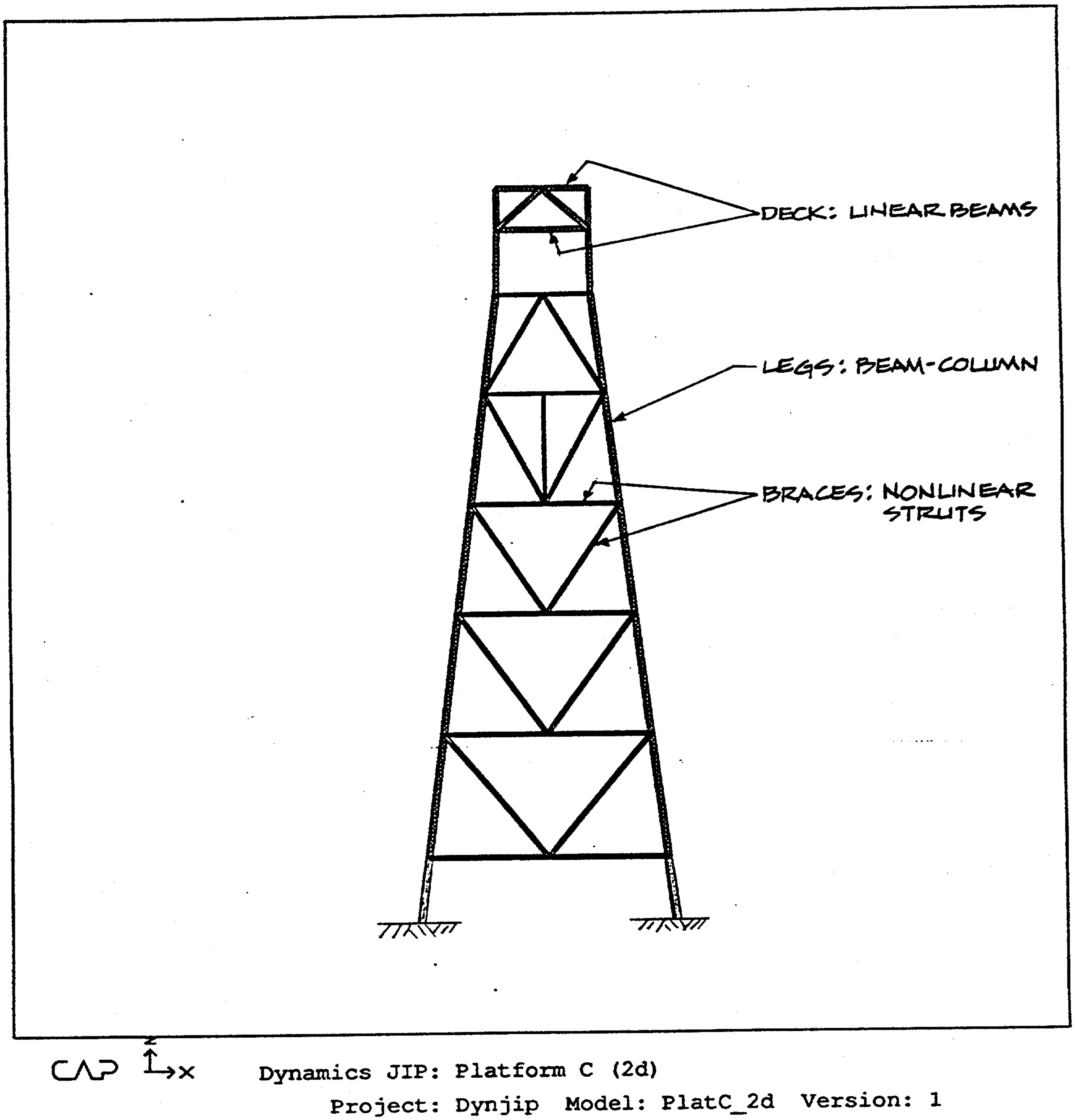
**DEFINITION OF MATERIAL INPUT PROPERTIES
FOR MARSHALL STRUT ELEMENT**

Figure 4 - 10



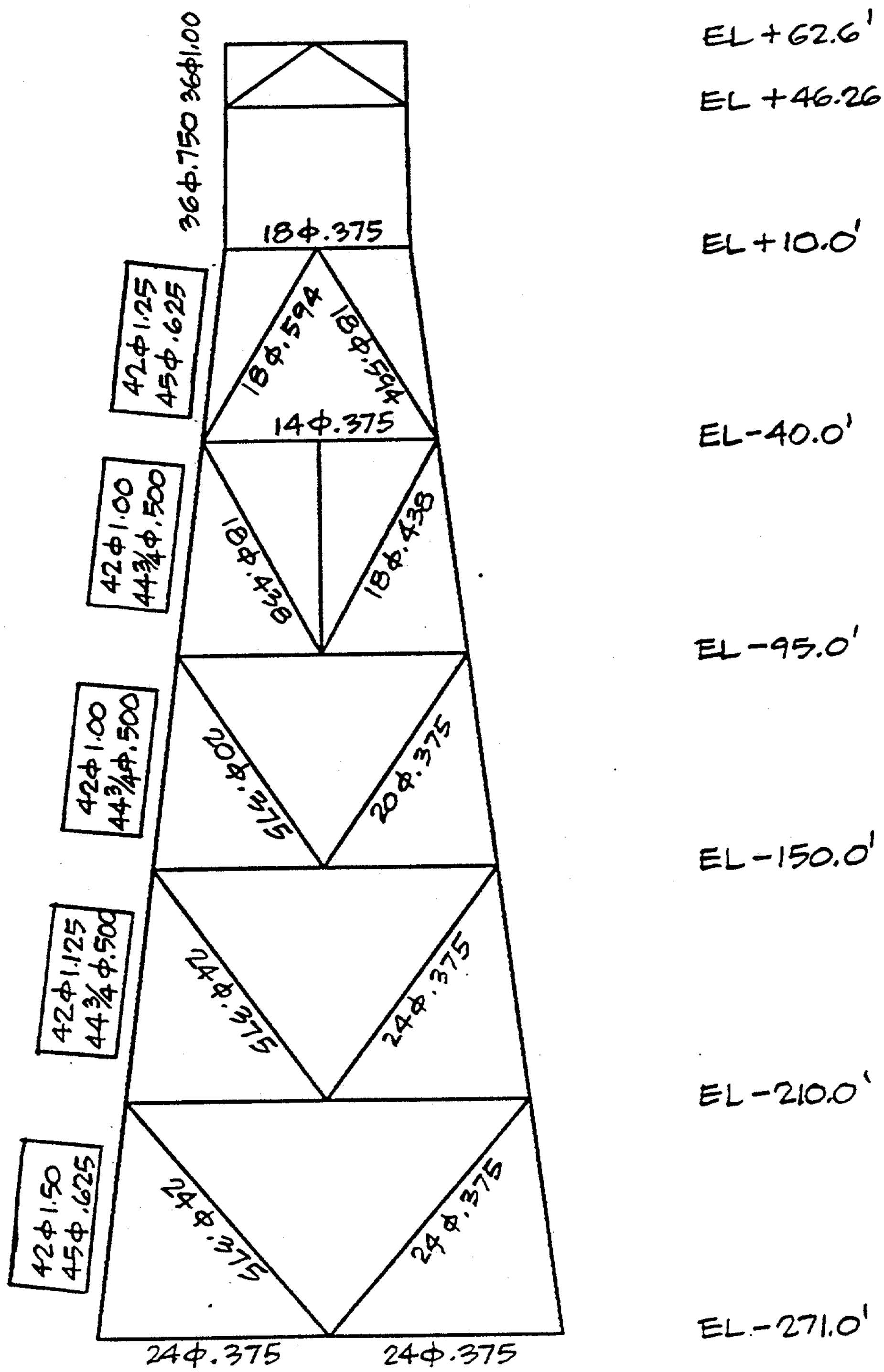
TYPICAL BRACE BUCKLING RESPONSE

Figure 4 - 11



2-D MODEL ELEMENTS

Figure 4 - 12



2-D MODEL MEMBER SIZES

Figure 4 - 13

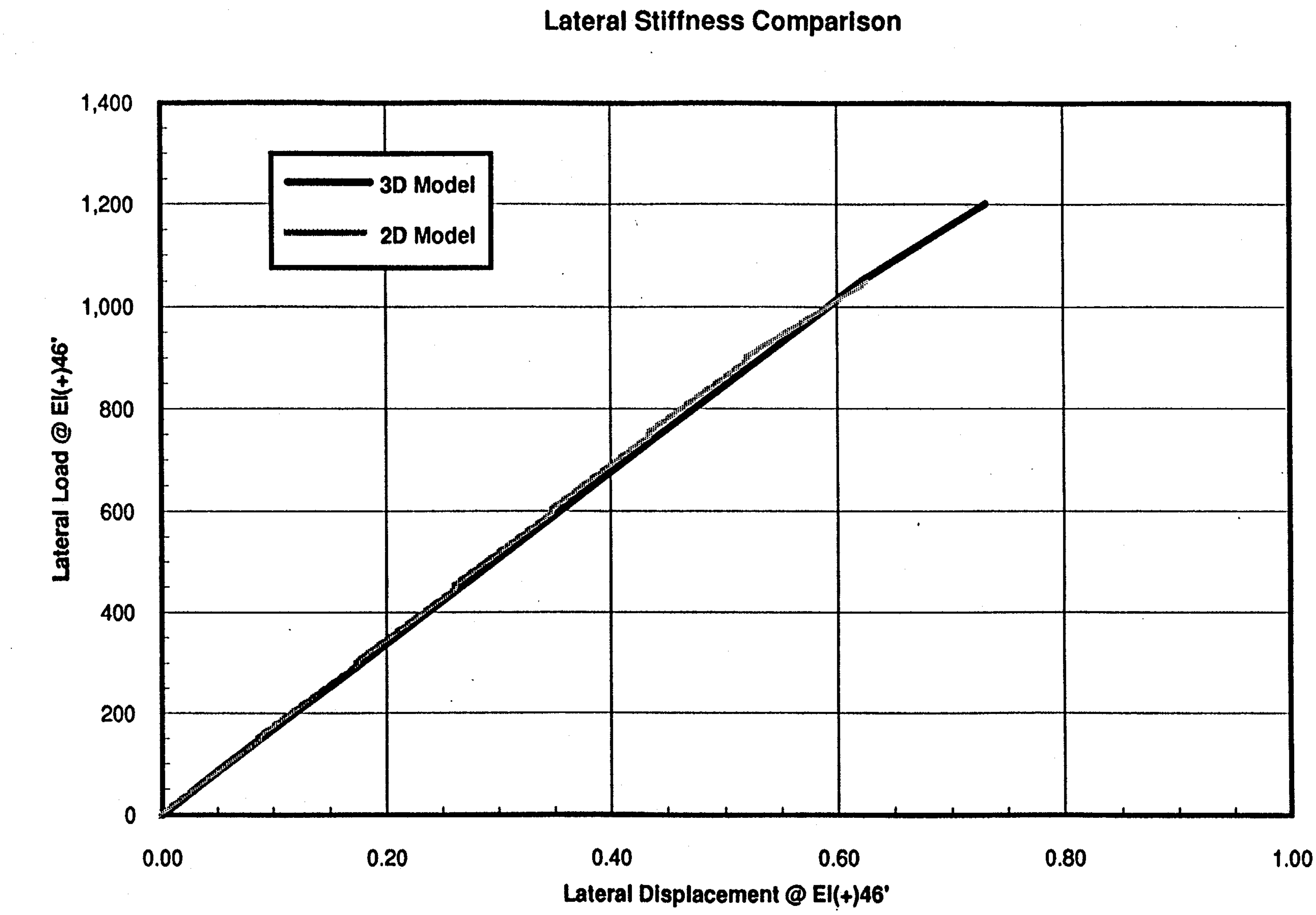
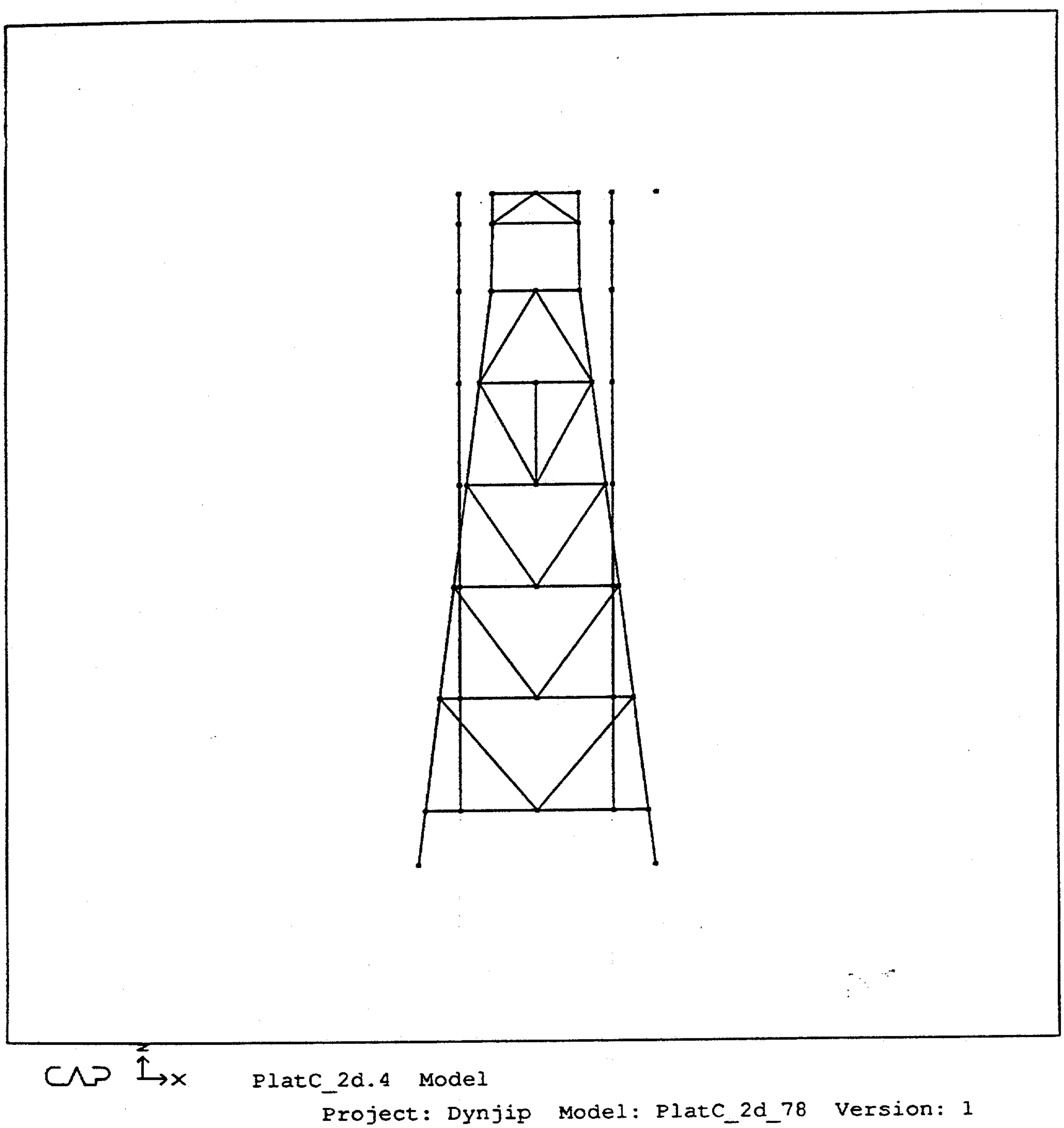
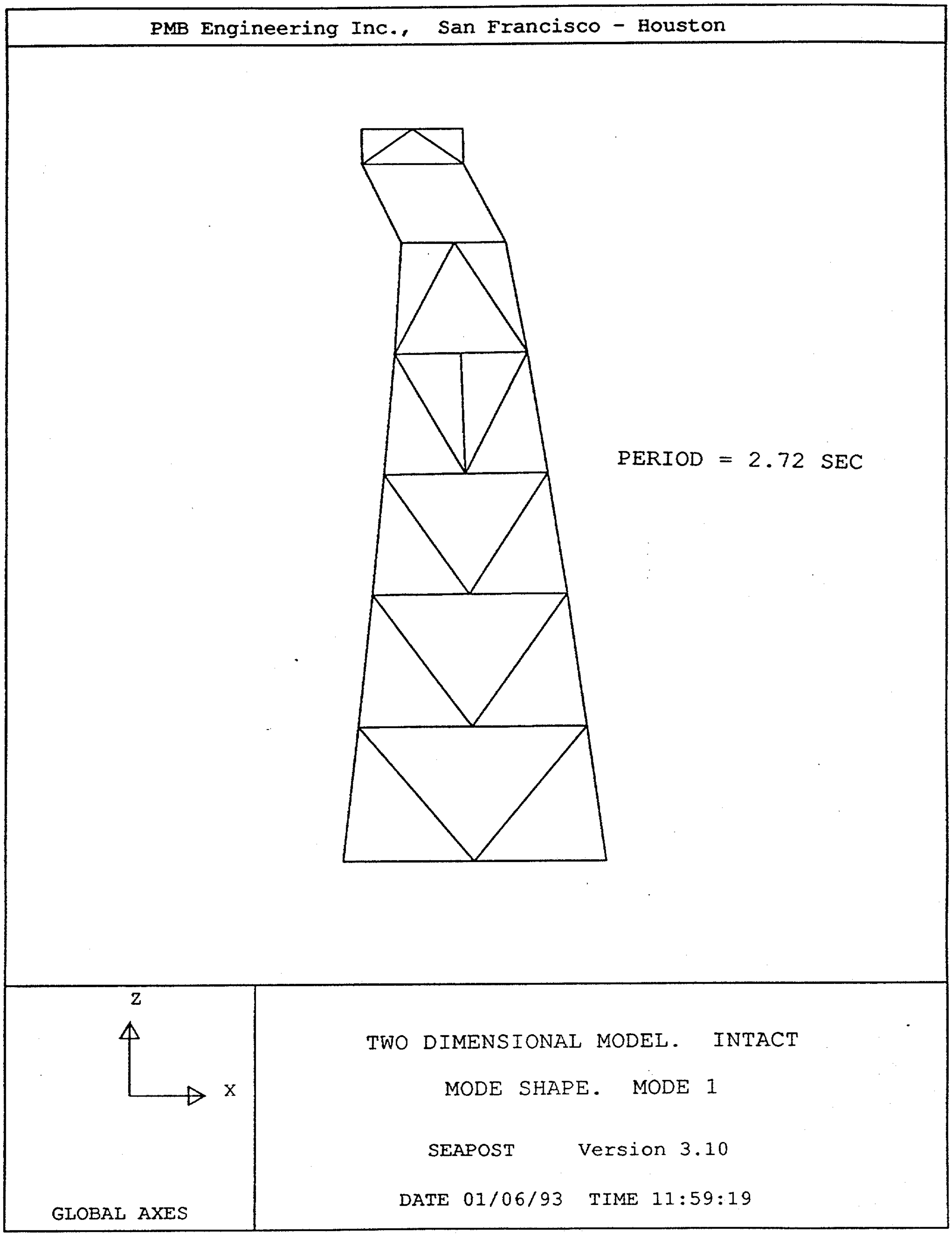

Lateral Stiffness Comparison
3D Model
2D Model
1,400
1,200
1,000
800
600
400
200
0
0.00
0.20
0.40
0.60
0.80
1.00
Lateral Load @ El(+)46'
Lateral Displacement @ El(+)46'

Figure 4 - 14



2-D MODEL

Figure 4 - 15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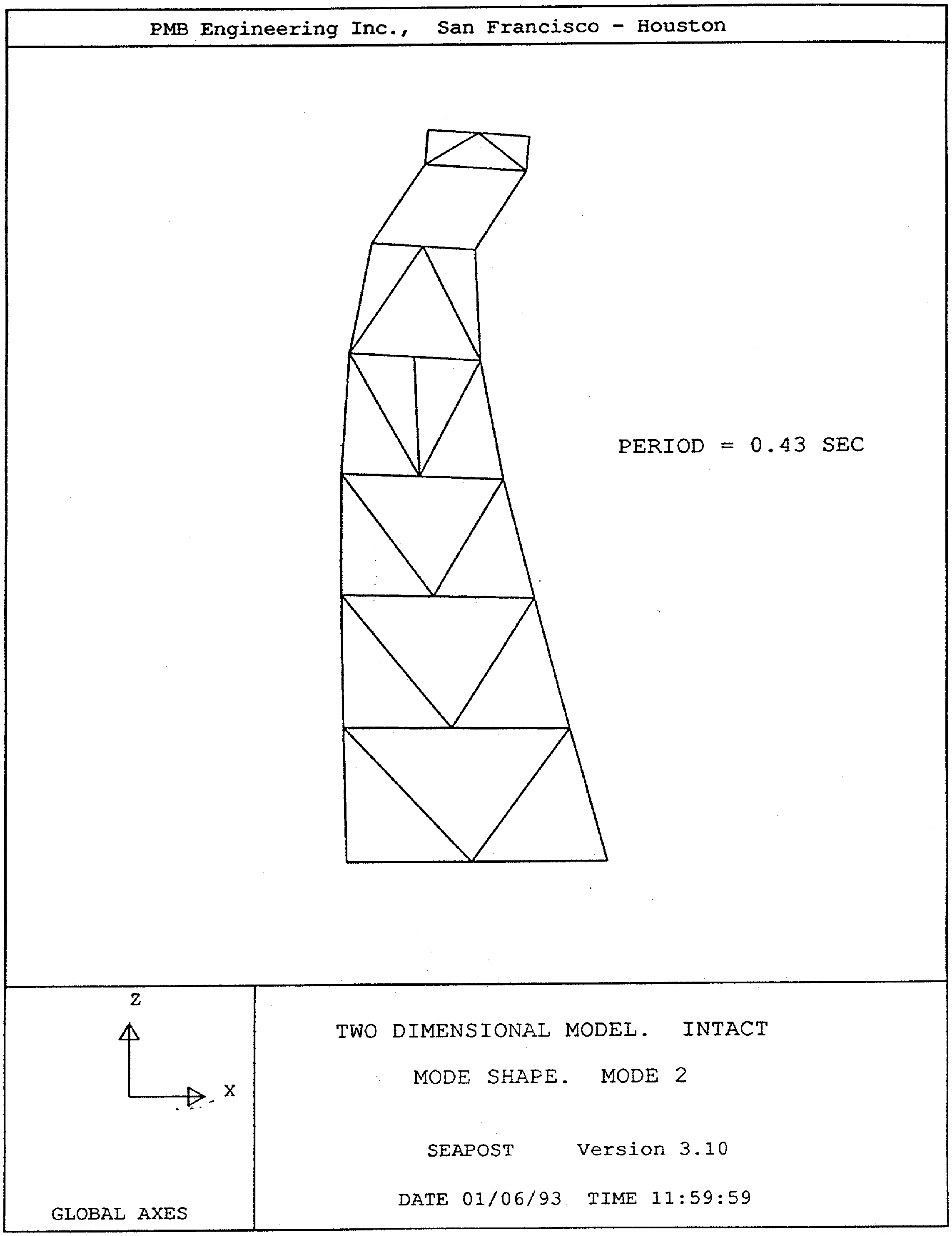
PERIOD = 2.72 SEC
Z
X
GLOBAL AXES
TWO DIMENSIONAL MODEL. INTACT
MODE SHAPE. MODE 1
SEAPOST Version 3.10
DATE 01/06/93 TIME 11:59:19

Figure 4-16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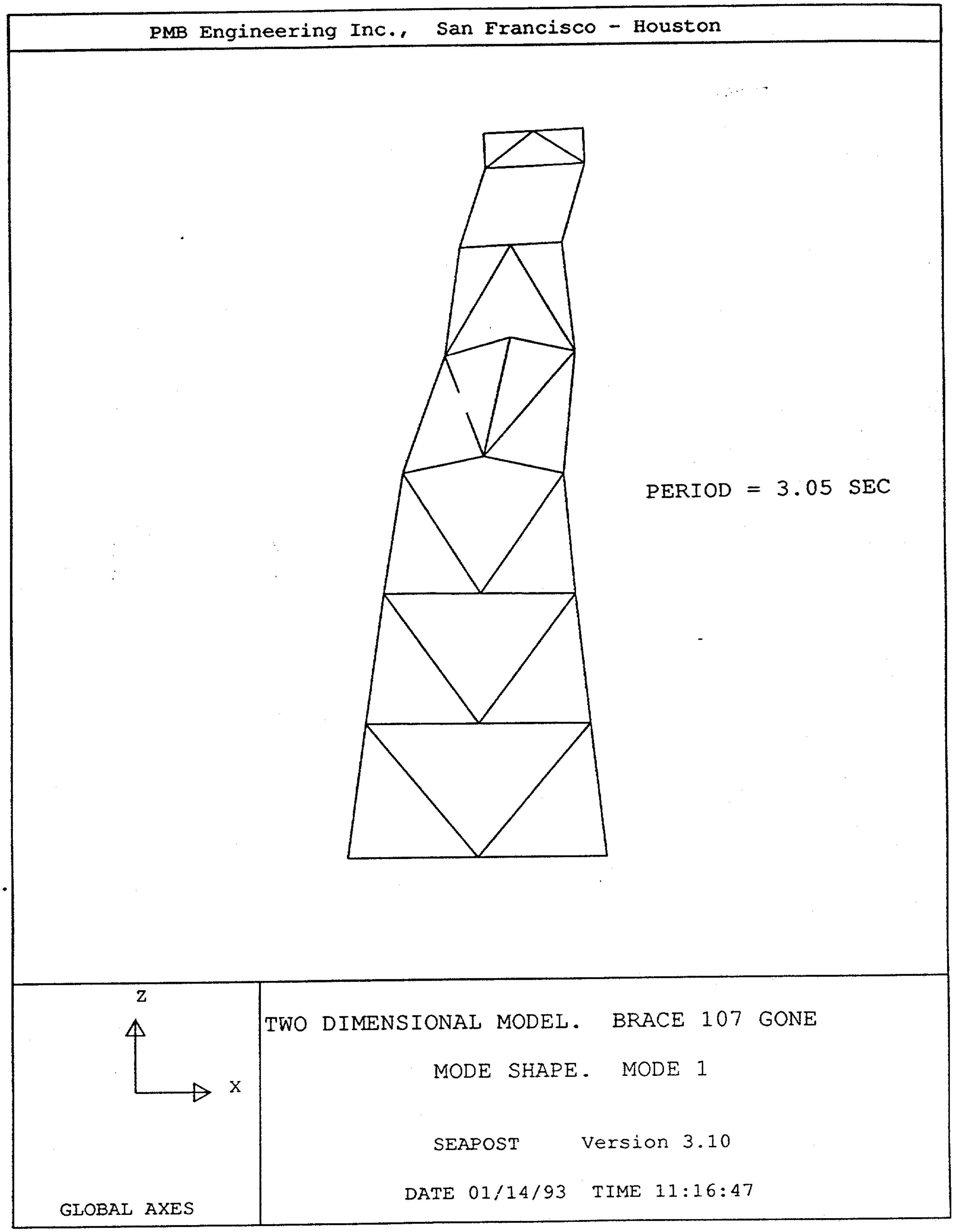
TWO DIMENSIONAL MODEL. INTACT
MODE SHAPE. MODE 2
SEAPOST Version 3.10
DATE 01/06/93 TIME 11:59:59

Figure 4-17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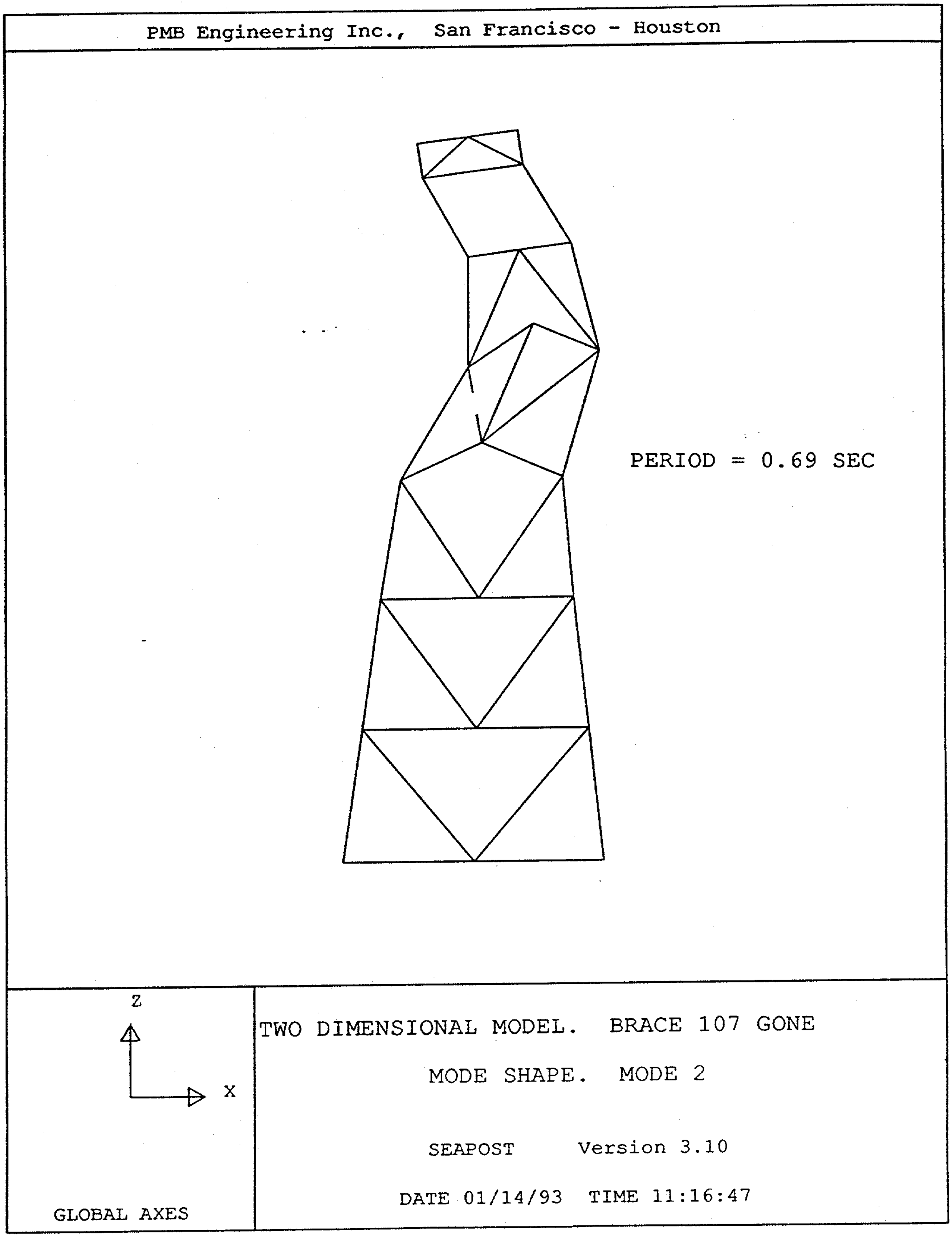
X
GLOBAL AXES
TWO DIMENSIONAL MODEL. BRACE 107 GONE
MODE SHAPE. MODE 1
SEAPOST Version 3.10
DATE 01/14/93 TIME 11:16:47

Figure 4-18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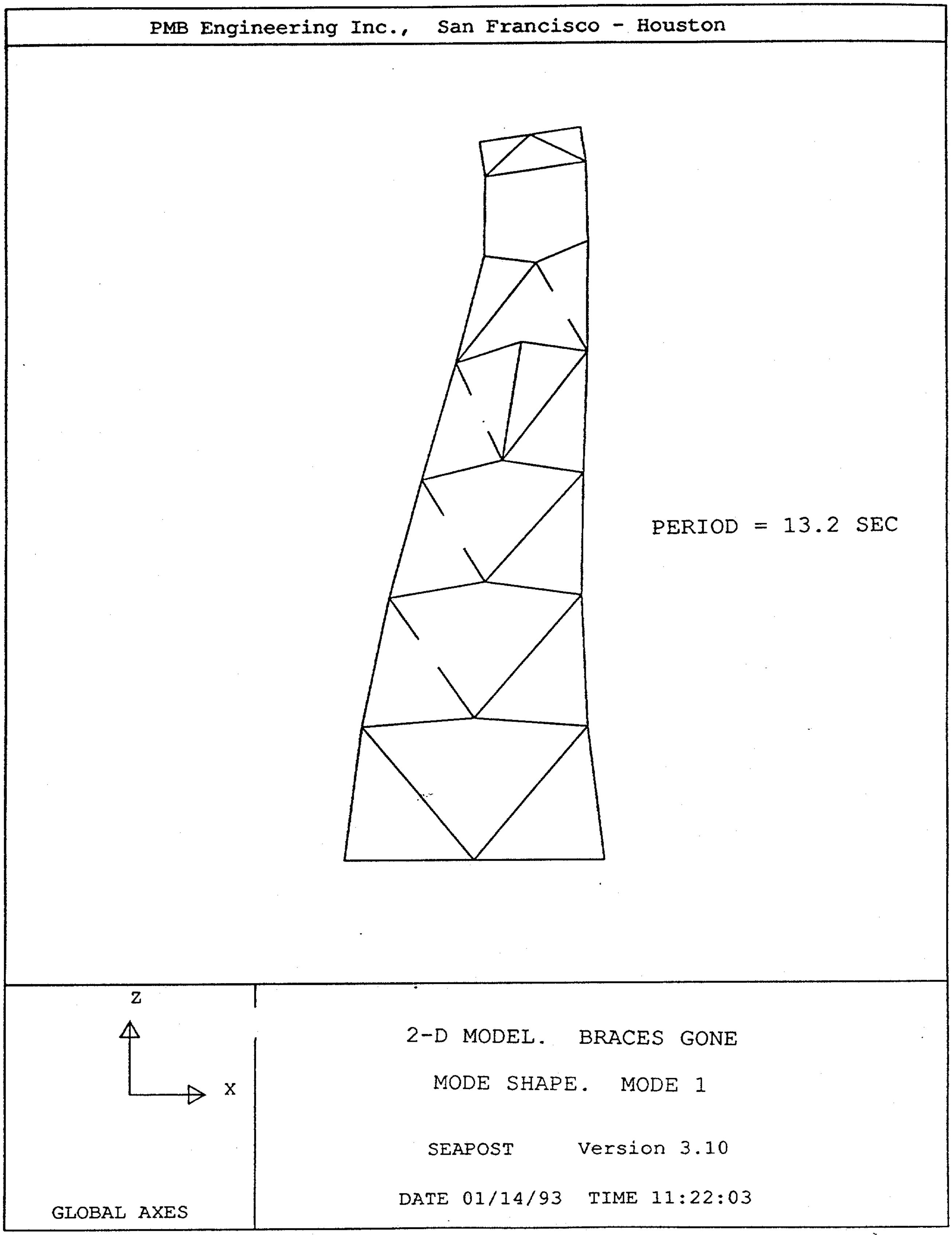
TWO DIMENSIONAL MODEL. BRACE 107 GONE
MODE SHAPE. MODE 2
SEAPOST Version 3.10
DATE 01/14/93 TIME 11:16:47

Figure 4-19


PMB Engineering Inc., San Francisco - Houston
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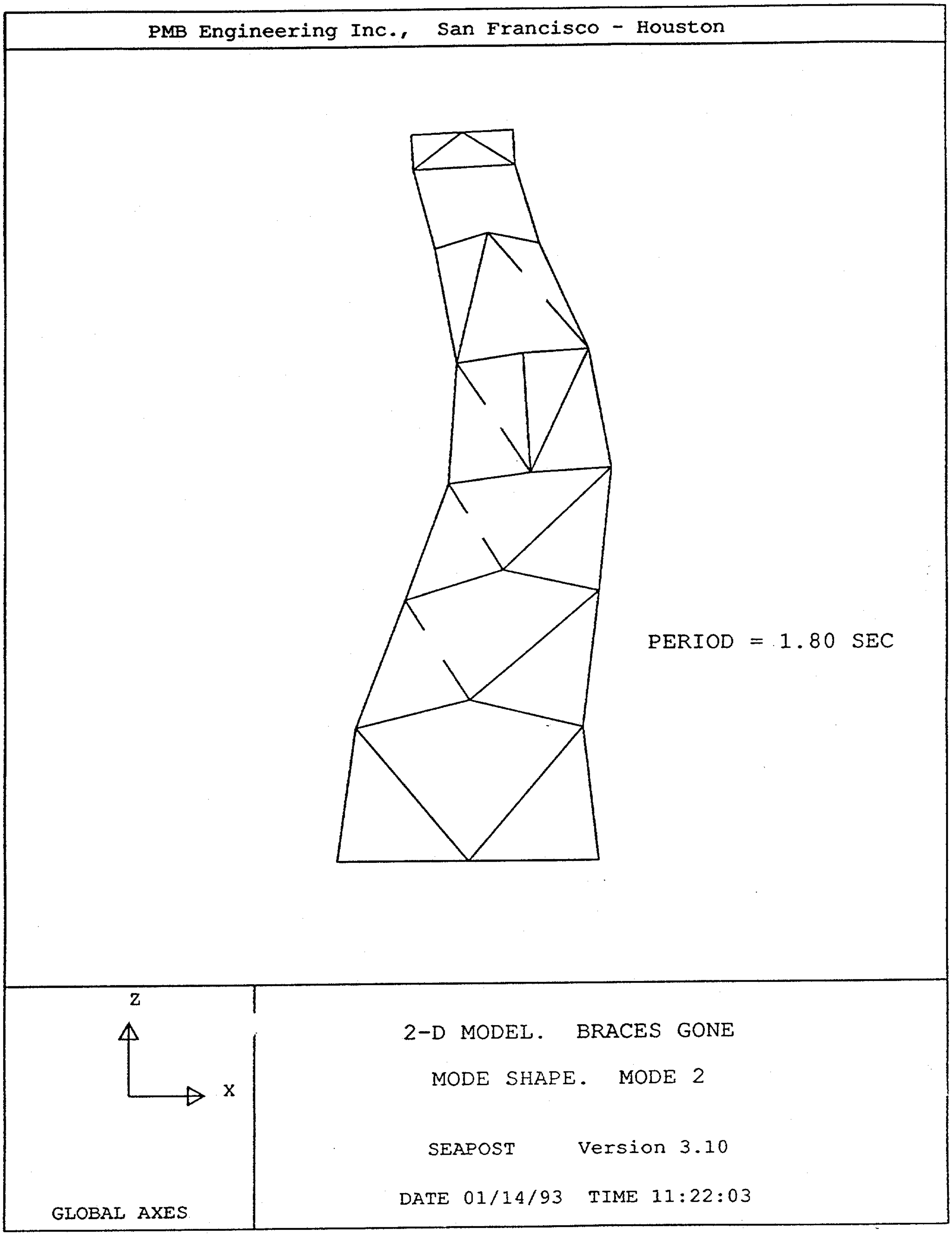
X
GLOBAL AXES
2-D MODEL. BRACES GONE
MODE SHAPE. MODE 1
SEAPOST Version 3.10
DATE 01/14/93 TIME 11:22:03

Figure 4-20


PMB Engineering Inc., San Francisco - Houston
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X
GLOBAL AXES
2-D MODEL. BRACES GONE
MODE SHAPE. MODE 2
SEAPOST Version 3.10
DATE 01/14/93 TIME 11:22:03

Figure 4-21

SHELL, API, AIM DECK LOADS

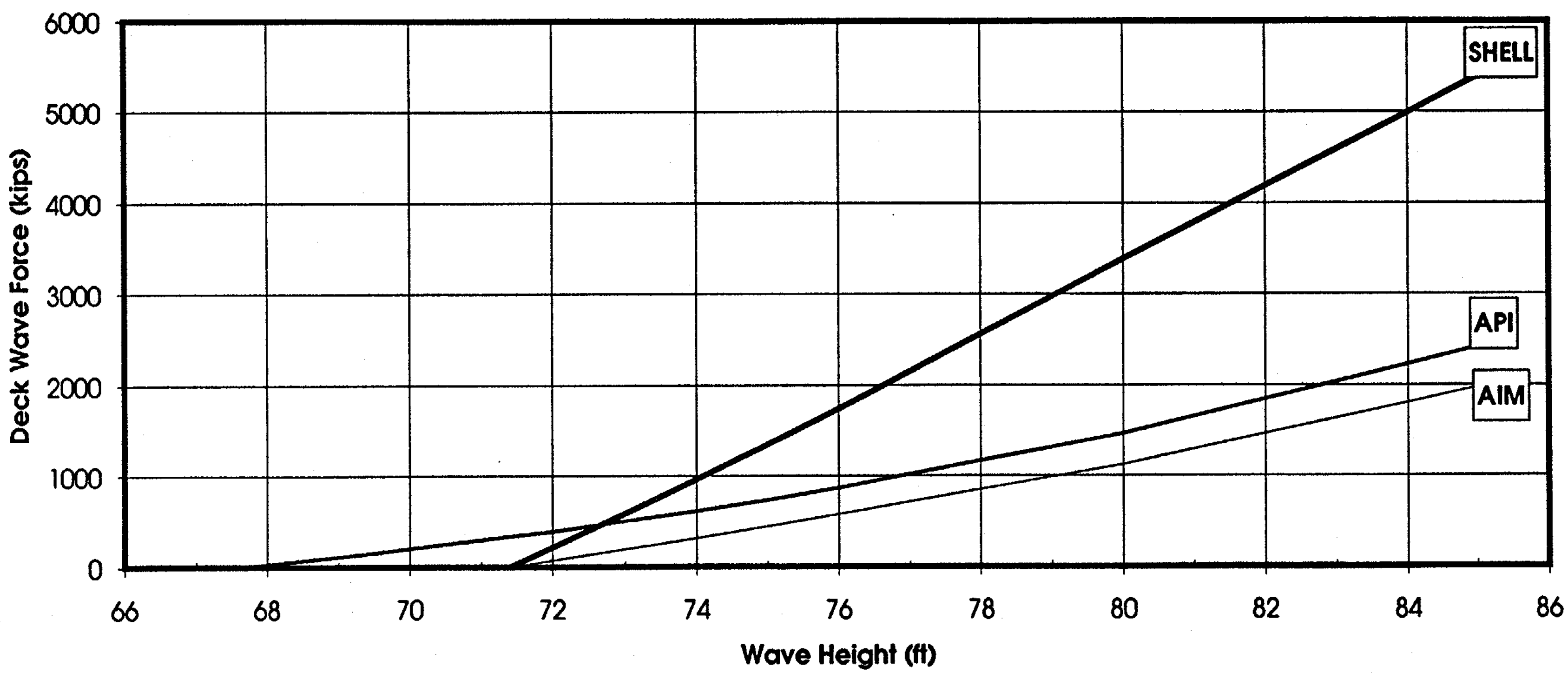


Figure 4-22a

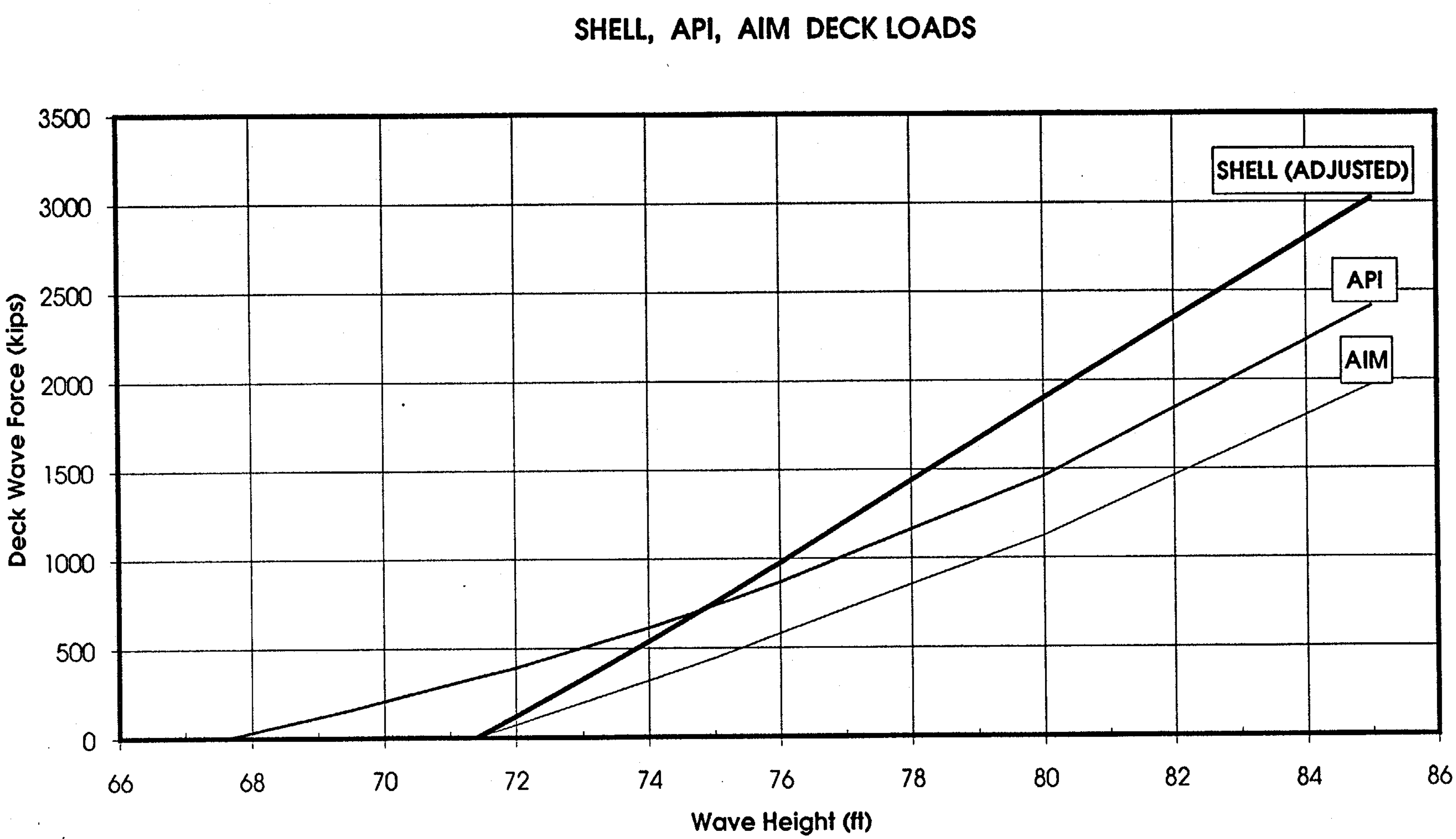
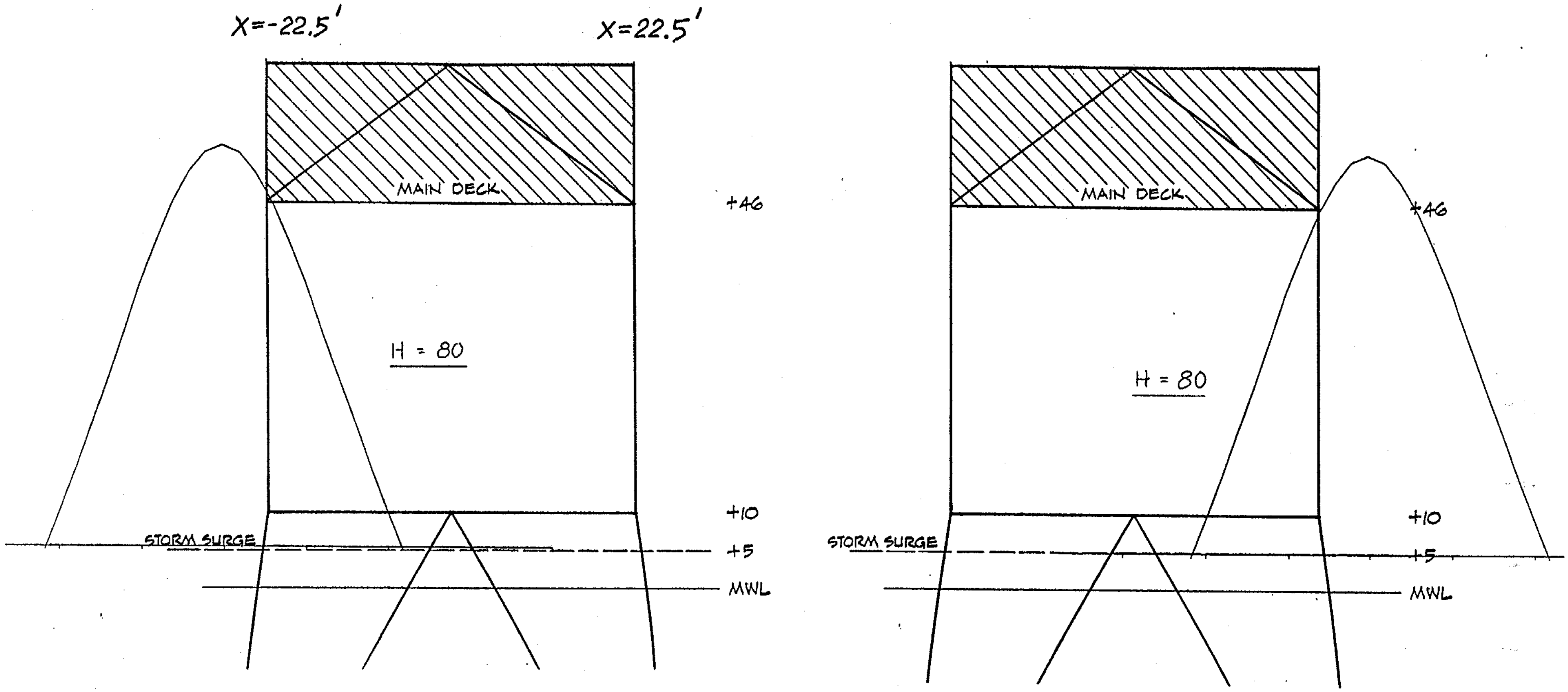

SHELL, API, AIM DECK LOADS
Deck Wave Force (kips)
3500
3000
2500
2000
1500
1000
500
0
66
68
70
72
74
76
78
80
82
84
86
Wave Height (ft)
SHELL (ADJUSTED)
API
AIM

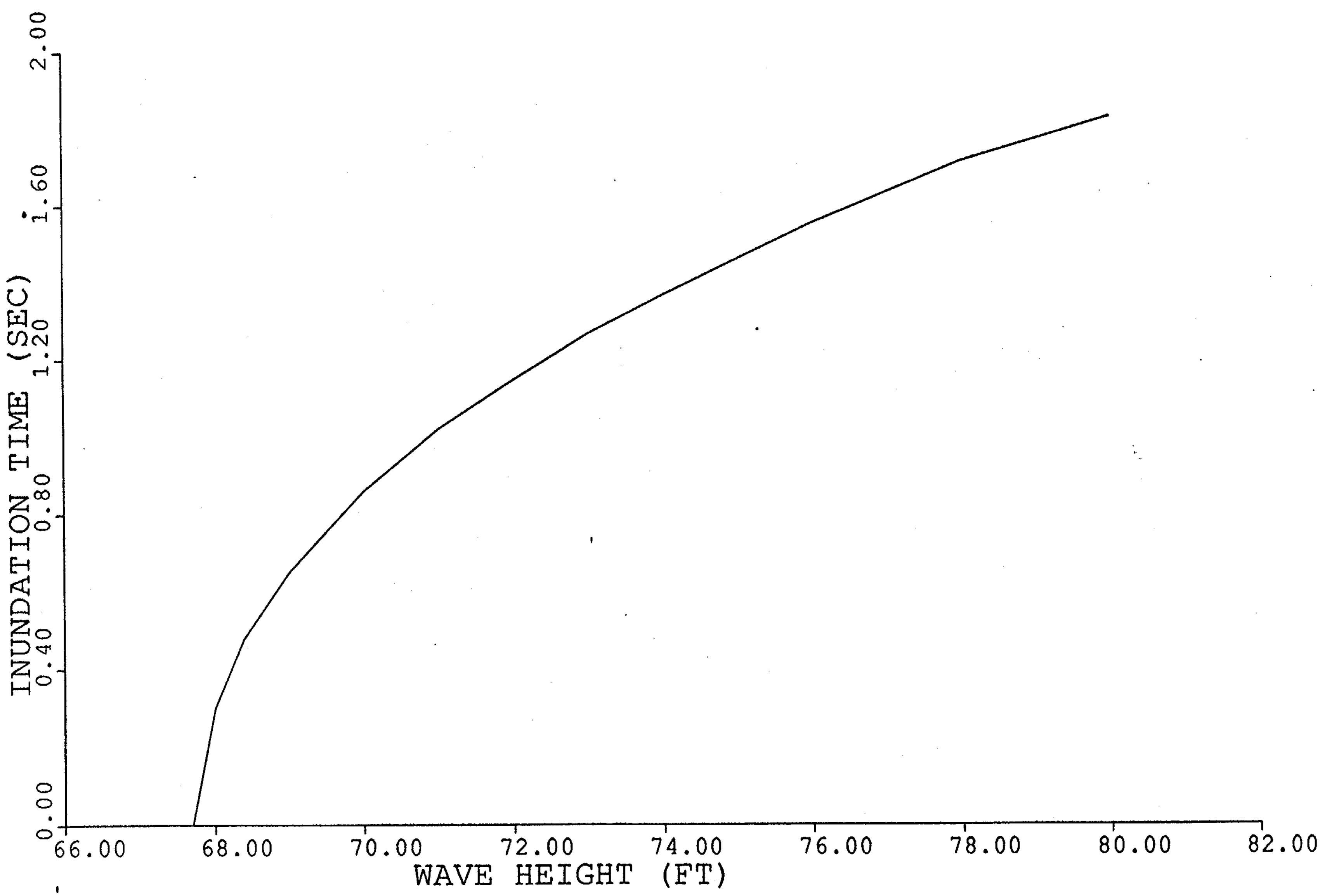
Figure 4-22b



A. Wave First Impacts Deck (t=0.0 sec)

B. Wave Leaves Deck (t=1.8 sec)

Figure 4 - 23



INUNDATION TIMES

Figure 4 - 24

Wave Forces in Deck
Broadside Direction -- No Current
(Ref.: Shell 1993)

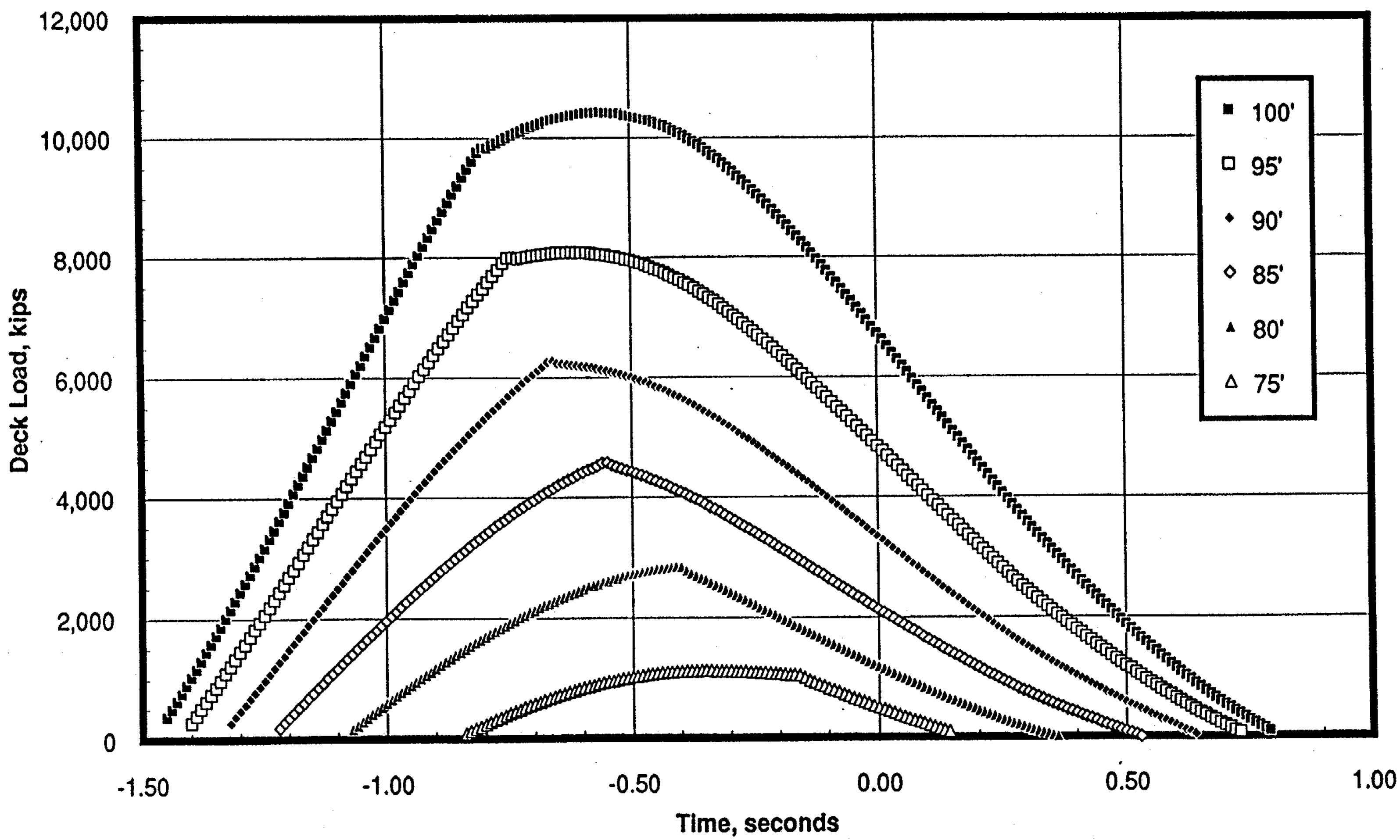


Figure 4 - 25

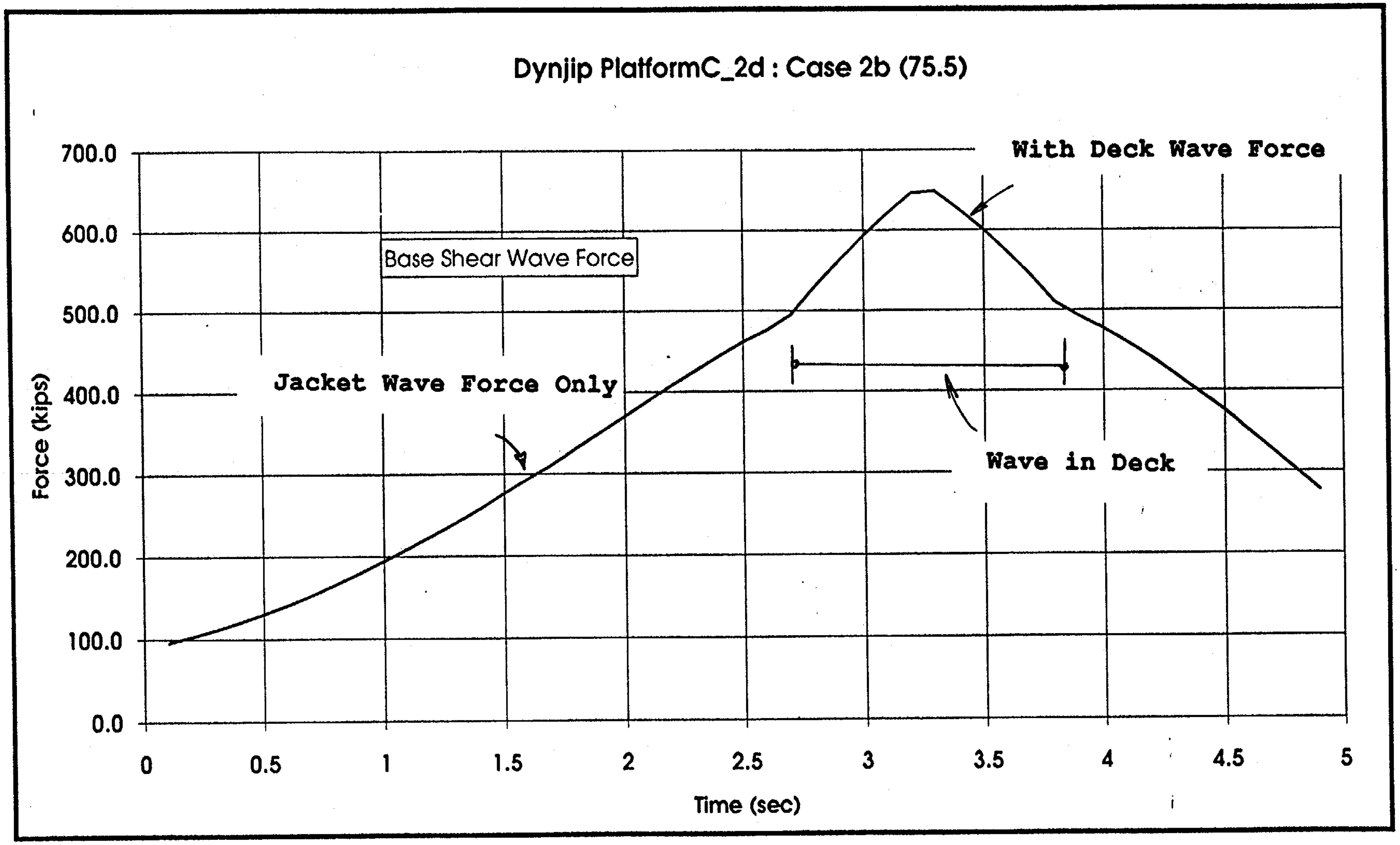


Fig. 4-26 Example Deck and Jacket Wave Force for Dynamic Analysis

Broadside Wave Force Comparison
API 17th vs. 20th Editions

(Wave Below Deck)

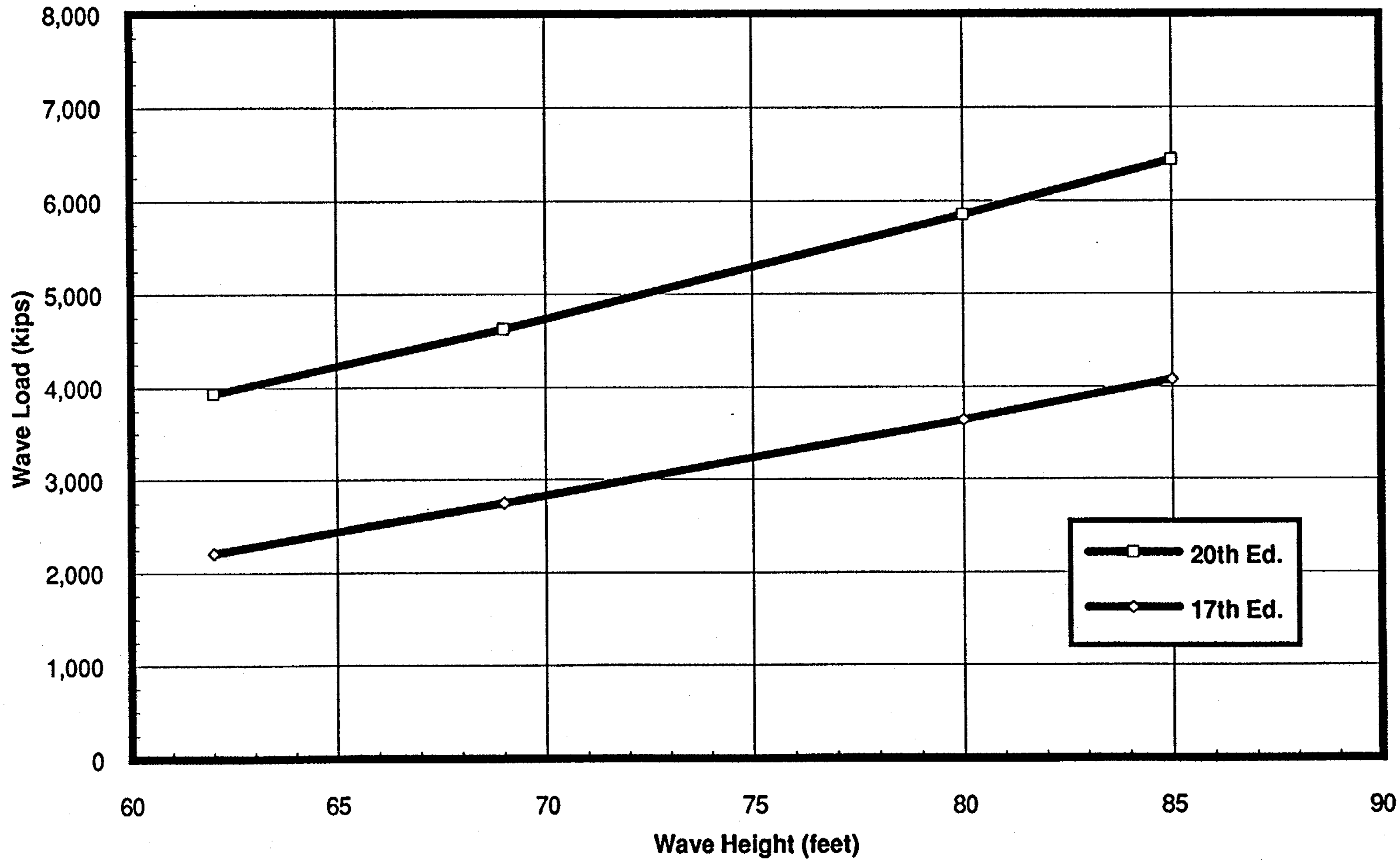


Figure 4 - 27

- Gulf of Mexico
- Main Pass Area - 271 ft. W.D.
- 8-Leg Template
- 24 Wells
- Self-Contained Drilling Platform
- Designed 1968-1969, Installed 1970
- Oil Producing
- Manned (Evacuated in Advance of Hurricanes)
- Eight 42-inch Piles
- Maximum Penetration 270 Feet
- Sands Overlying Stiff Clays
- Designed with 1969 Wave Criteria
- Lower Deck at +45 Ft
- Sump Deck at +35'-6"
- Deck Weight = 600 Tons (AIM)
- Topsides Weight = 5500 Tons (AIM)

2500 Tons (This project)

- Risers 1 at 16" Diameter
  2 at 10" Diameter
- Pump Casings 3 at 16" Diameter
- Two Boat Landings
- Barge Bumpers
- Sacrificial Anodes Corrosion Protection

PLATFORM "C" KEY DATA

Table 4-1

Table 4-2
Platform C Natural Periods

Mode	Direction	6100 Ton Deck & Equip	2500 Ton Deck and Equipment		
			Full Fnd.	Fixed Base	Deck Only
1	X-Sway	3.07 sec	2.08 sec	1.81 sec	1.15 sec
2	Y-Sway	3.01 sec	2.08 sec	1.80 sec	1.01 sec
3	Torsion	2.49 sec	1.68 sec	1.60 sec	1.01 sec

Structural Period Variation of Damaged Structure

End on Loading						
Mode	Direction	Failed Members 0 Intact	 1 Case 1	 2 Case 2	 4 Case 3	 8 Case 4
1	Y Sway	3.072	3.074	3.074	3.076	3.092
2	X Sway	3.010	3.027	3.048	3.107	3.432
3	Torsion	2.488	2.497	2.504	2.527	2.615

Broadside Loading						
Mode	Direction	Failed Members 0 Intact	 6 Case 1	 9 Case 2	 14 Case 3	 20 Case 4
1	Y Sway	3.072	3.409	3.864	5.315	7.470
2	X Sway	3.010	3.009	3.009	3.010	3.009
3	Torsion	2.488	2.604	2.671	3.075	3.840

Note: Member Failure Sequence per AIM III

Table 4-3
Periods of Intact and Damaged 3-D Structure

Comparision of API 17th Ed. and API 20th Ed. Wave Loading Factors

	API 17th Ed.	API 20th Ed.
0. Wave Height (100 year RP)	69'	68'
1. Apparent Wave Period	12.7 sec.	13.5 sec.
2. 2-D Wave	Stokes V	Stokes V
3. Kinematics Reduction Factor	none	0.88
4. Current Blockage Factor	none	0.7 (X dir) 0.8 (Y-dir)
6. Marine Growth	2" on dia. (0 to -100')	3" on dia. (0 to -150')
7a. Cd	0.7	0.65 (smooth) / 1.05 (rough)
7b. Cm	1.5	1.60 (smooth) / 1.20 (rough)
8. Conductor Shielding	none	X-dir. 0.95 (MG) 0.97 (No MG) Y-dir: 0.95 (MG) 1.00 (No MG)
9. Hydrodynamic Models for Appurtenances	yes	yes
10. Morison Equation	yes	yes
11. Global Structure Force	yes	yes

Table 4-4

Table 4-5 Hydrodynamic Coefficients

	Drag Coefficient	Inertia Coefficient
Above -150 ft	1.05	1.2
Below -150 ft	0.65	1.6

	3-D Model	2-D Model
First Mode	2.14 sec	2.08 sec
Second Mode	0.92 sec	0.90 sec

Comparison of Natural Frequencies, 3-D and 2-D Models

Table 4-6

Preliminary Deck Force Guidelines

The purpose of these guidelines is to provide a simple yet conservative method for predicting the wave loads on fixed platform decks for use in the "Task Group on Assessment of Existing Platforms to Demonstrate Fitness for Purpose." The method is presented as an interim method for the purpose of evaluating the three-tiered screening procedure for assessing fixed offshore platforms. The procedure returns the maximum wave-induced deck load, which is assumed to occur at the same time as the maximum base shear, and the moment induced by the deck load. The steps for computing the base shear and overturning moment caused by the deck follows. This procedure is expected to return a relatively conservative deck force and moment, with a COV on the order of 35%.

1. Given the crest height (see note below), compute the wetted "silhouette" deck area, A, projected in the wave direction, θ_w. The silhouette area is defined as the shaded area in Figure 1. The area, A, is computed as:

   $$A = A_x \cos\theta_w + A_y \sin\theta_w,$$

   where θ_w, A_x and A_y are as defined in Figure 2.

   For lightly framed "sub-cellar" deck sections with no equipment, such as a "scaffold" deck comprised of angle iron, use one-half of the silhouette area. The areas of the deck legs and bracing above the cellar deck are part of the silhouette area. Deck legs and bracing members below the bottom of the cellar deck should be computed along with jacket members in the jacket force calculation procedure.

2. Use Stream Function Wave Theory or equivalent with specified wave period, water depth, and current speed to compute the maximum horizontal fluid velocity, V, at the crest elevation or the top of the deck structure, whichever is lower. A directional spreading factor of 0.88 is applied to the velocity.

3. The wave force on the deck, F, is computed as follows:

   $$F = 1/2\, \rho\, C_D (V + \alpha U)^2 A$$

   where U is the current velocity, with the same blockage factor, α, as specified for the jacket. The drag coefficient, C_D, is given in the table below:

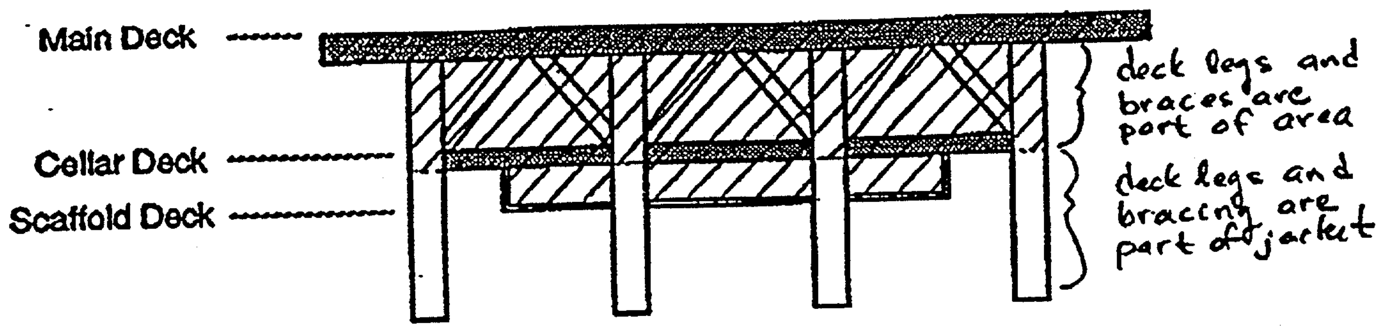
Deck Type	end-on and broadside	oblique (45-degrees)
modern deck (very dense)	2.5	1.9
heavily equipped "older" deck	2.0	1.5
bare "older" deck	1.6	1.2

4. The overturning moment on the jacket due to wave loads on the deck is obtained by applying the deck load to a point 60% of the way between the lowest point of the silhouette area and the lower of the wave crest or top of deck.

note: The above procedure relies on the use of an adjusted wave height when using Stream Function Wave Theory (or equivalent) in order to return crest elevations that are in closer agreement with measured data. The wave heights for the deck force calculation should be 1.056 times the wave heights used for the jacket force calculation.

API Preliminary Deck Force Guidelines (April 30, 1993)

Table 4-7



Elevation View of Platform Deck

Figure 1: Silhouette Area Definition.

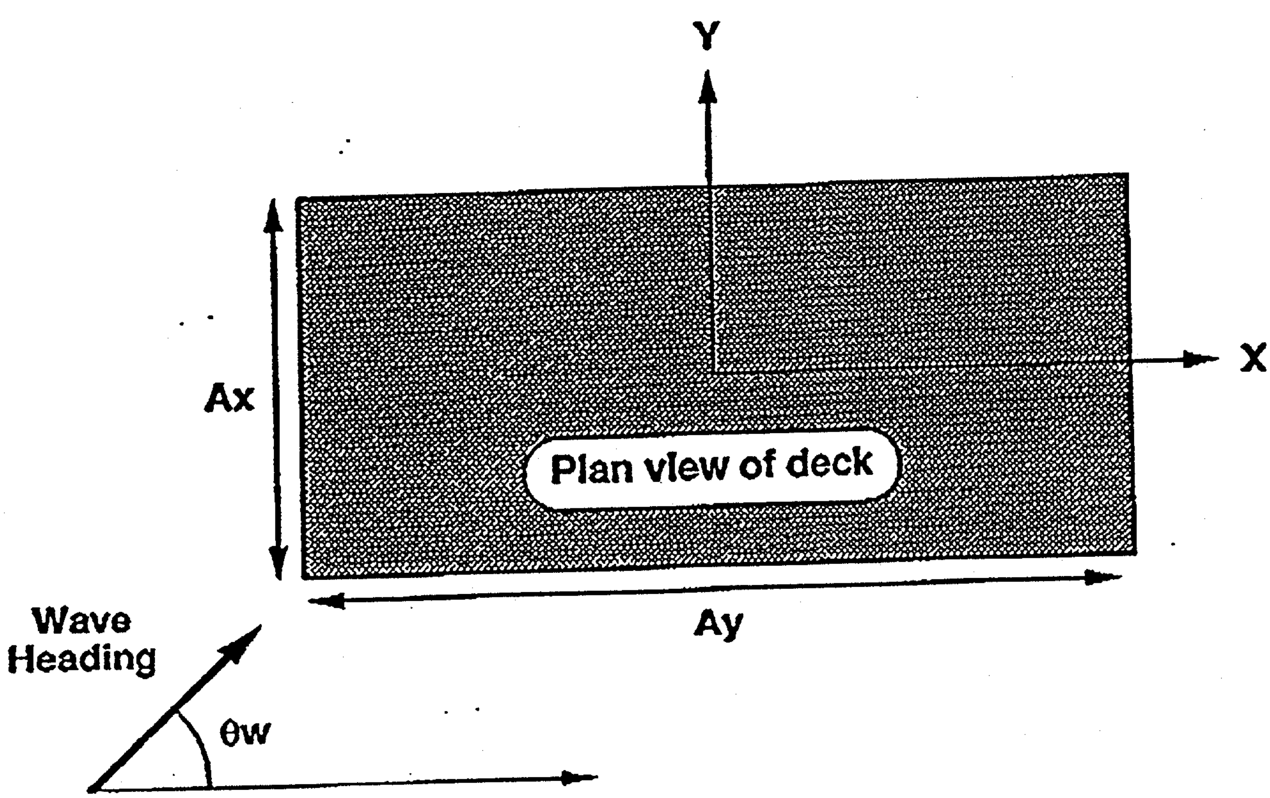


Figure 2: Wave heading and Direction Convention.

Table 4-7 (cont)

Wave Force Comparison Summary

API 17th & 20th Editions

ITEM		% Change
Wave Force Kinematics Factor	=	-23% (1.00 to 0.88)
Drag Coefficients	=	25% (0.70 to 1.05)
Marine Growth	=	6% (2" to 3"; 100' to 150')
Addition of Current (w/ blockage)	=	60% (0.0 ft/sec to 2.8 ft/sec)
TOTAL	=	68%

Table 4-8

Section 5
Static Capacity

5.1 APPROACH

The static pushover procedure used in this project is similar to that used in the AIM projects and typically used in the industry for platform re-assessment. The following paragraphs summarize the procedure for those not familiar with the approach. The remaining portion of this section summarizes the base case static pushover results for the two and three dimensional Platform C computer models. These static capacities provide a reference level for later comparison with dynamic capacity.

The step-by-step static pushover approach used in this project was as follows:

1. **Develop nonlinear computer model.** Section 4 has previously described the nonlinear computer models used for the analysis. The models reflect inelastic behavior as individual members attain their maximum load carrying capabilities. The models also have hydrodynamic member properties which allow an estimate of wave and current forces acting on the platform.

2. **Develop relationship of base shear versus wave height.** A series of different waves were run past the platform and the maximum base shear recorded for each. The current associated with each wave was also included where appropriate. For static pushover, the wave forces on the deck were not explicitly accounted for in the computer model; they were instead determined by hand calculations (see Section 4.4) and then added as static loads to the wave and current loads acting on the jacket. Wind loads were computed by hand and added in a similar manner. Figure 5-1 shows the resulting wave height versus maximum base shear relationship for Platform C using the Shell deck wave forces.

3. **Develop static pushover load profile.** An estimate is made of the base shear that will fail the platform (e.g. 4800 kips). For Platform C, this information was available from previous work on the AIM projects. The base shear versus wave height relationship (Figure 5-1) was then used to determine the height of the wave (e.g. 74 ft, wave-in-deck condition) that results in this level of base shear. This wave was then again run past the platform, the hydrodynamic forces acting on each platform member (or node) at the time of peak base shear was stored to create a "snapshot" in time of the lateral hydrodynamic forces acting on the platform at the peak base shear for this size wave. Figure 5-2 shows the load profile acting on Platform C for the 74 ft. wave-in-deck condition.

4. **Initialize analysis with gravity forces.** The platform computer model is first initialized by adding gravity forces (deadweight and buoyancy). These forces remain constant throughout the analysis.

5. **Apply pushover load.** The pushover lateral load profile (Figure 5-2) is then applied in a step-by-step incremental manner (often called load ramping). Recall that the pushover load profile contains the "snapshot" wave and current forces, plus the static deck wave and deck wind forces. For Platform C, the load was applied in 0.5 percent increments until the platform failed. The member failure sequence and ultimate capacity were recorded. Figure 5-3 shows the static pushover results up to the point of reaching ultimate capacity for Platform C, wave-in-deck condition.

6. **Determine wave height that will cause failure.** The platform capacity (Figure 5-3) is compared to the wave height versus base shear relationship (Figure 5-1) to determine the wave height that will cause platform failure.

7. **Compare pushover wave with the failure wave.** If the wave causing failure differed considerably (say more than 10 percent of base shear) from the wave used for pushover, then the pushover analysis was repeated with a new failure wave with height based upon the wave causing failure in the pushover analysis. The process typically converged in 2 to 3 iterations. This iterative process ensured that at time of failure, the distribution of forces on the platform (i.e. those acting on the jacket and those acting on the deck) were proportioned appropriately for a wave of that magnitude.

5.2 STATIC VERSUS PSEUDO-STATIC PUSHOVER

The CAP computer code contains several automatic features which simplify several of the above steps. CAP also uses a "pseudo static" approach for estimating platform capacity. CAP's pseudo static approach is actually a dynamic analysis with special dynamic control parameters that allows the platform to incur significant deflections while still remaining stable. This method is used since it is more robust (i.e. ability to achieve a solution), requires less user interaction and is less time consuming than a static pushover. The pseudo static pushover determines only the platform's ultimate capacity, and not the post peak capacity response. Since the platform capacity is the prime concern for pushover analysis, the pseudo static approach was adequate for the work of this project.

In order to ensure the pseudo static approach was properly reflecting a static pushover for Platform C, a two-dimensional static pushover (also using CAP) was performed and compared to results from a two-dimensional pseudo static pushover of the same configuration. Figures 5-4 and 5-5 show results of the analysis. The capacities and member failure sequence identified by each of the analysis are identical. The major difference is the post peak capacity which is defined by the static approach but not the pseudo static approach. Other comparisons of the static and pseudo-static analysis run for other projects and on in-house PMB studies have also shown that the capacities computed by the two methods are essentially identical.

5.3 STATIC CAPACITY OF PLATFORM C — BASE CASE

5.3.1 Wave-Below-Deck

Figure 5-4 shows results of the static pushover for the two-dimensional Platform C computer model, base case condition, wave-below-deck, broadside direction. Figure 5-6 shows results for the two-dimensional model for the same conditions. The results are very similar with both models predicting a failure wave height of about 80 feet, equivalent to a base shear of about 4,800 kips for the three-dimensional model. The pushover analysis was iterated several times until the difference between the pushover wave profile and the failure base shear were less than 5 percent.

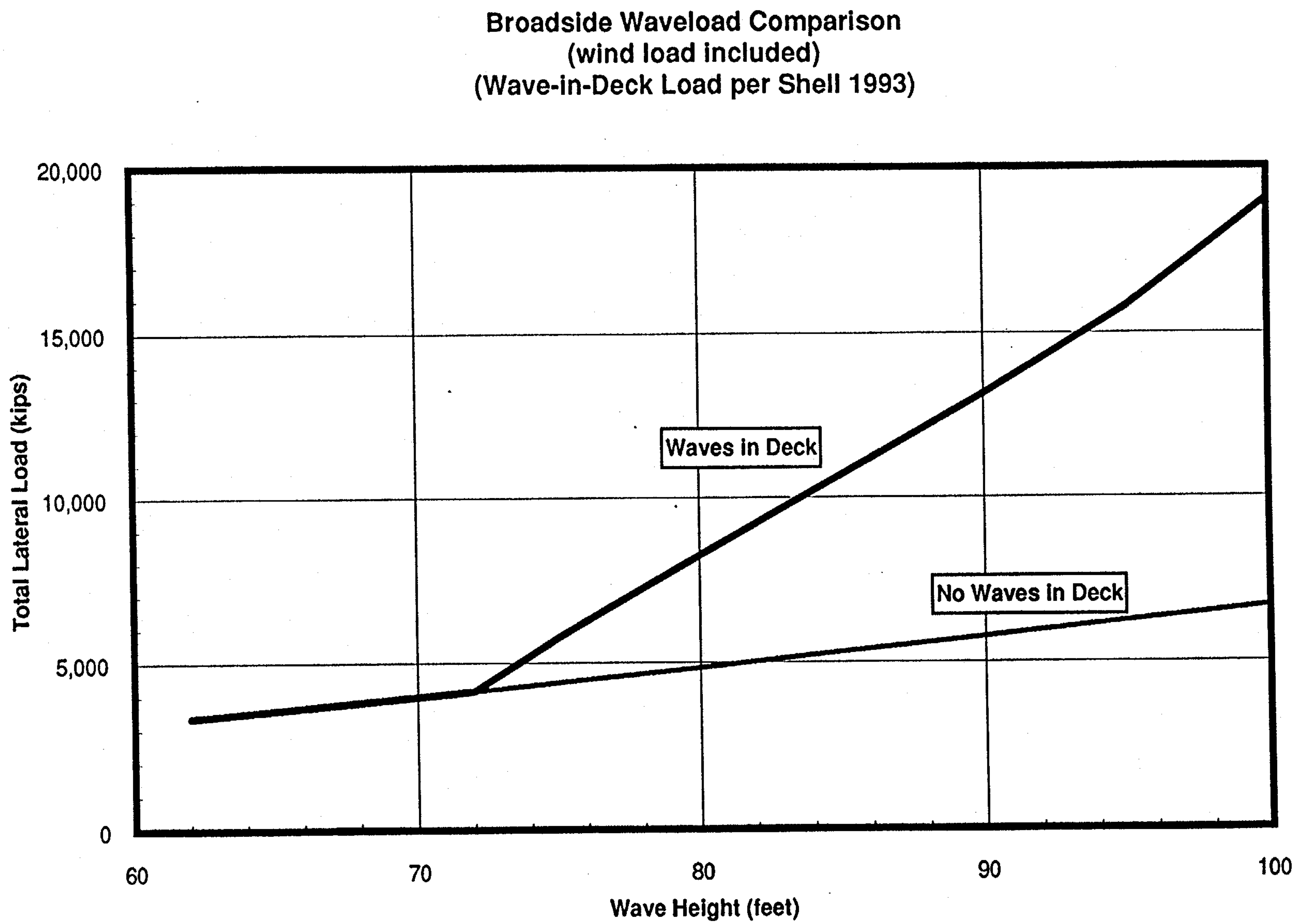
Also shown on Figures 5-4 and 5-6 are the member failure sequences. The first member failure is brace failure in the k framing just below the water line, followed by additional brace failures, until yielding occurs at the legs. The collapse mechanism is double hinging of the legs at the elevations where the brace failures occur.

Figures 5-7, 5-8 and 5-9 show snapshots of the member failure sequence for the two-dimensional model. Figures 5-10, 5-11 and 5-12 show snapshots of the member failure sequence for the three-dimensional model. It is clear that the failure sequences are almost identical, indicating that the two-dimensional model is a good representation of the three-dimensional model for static pushover analysis.

5.3.2 Wave-In-Deck

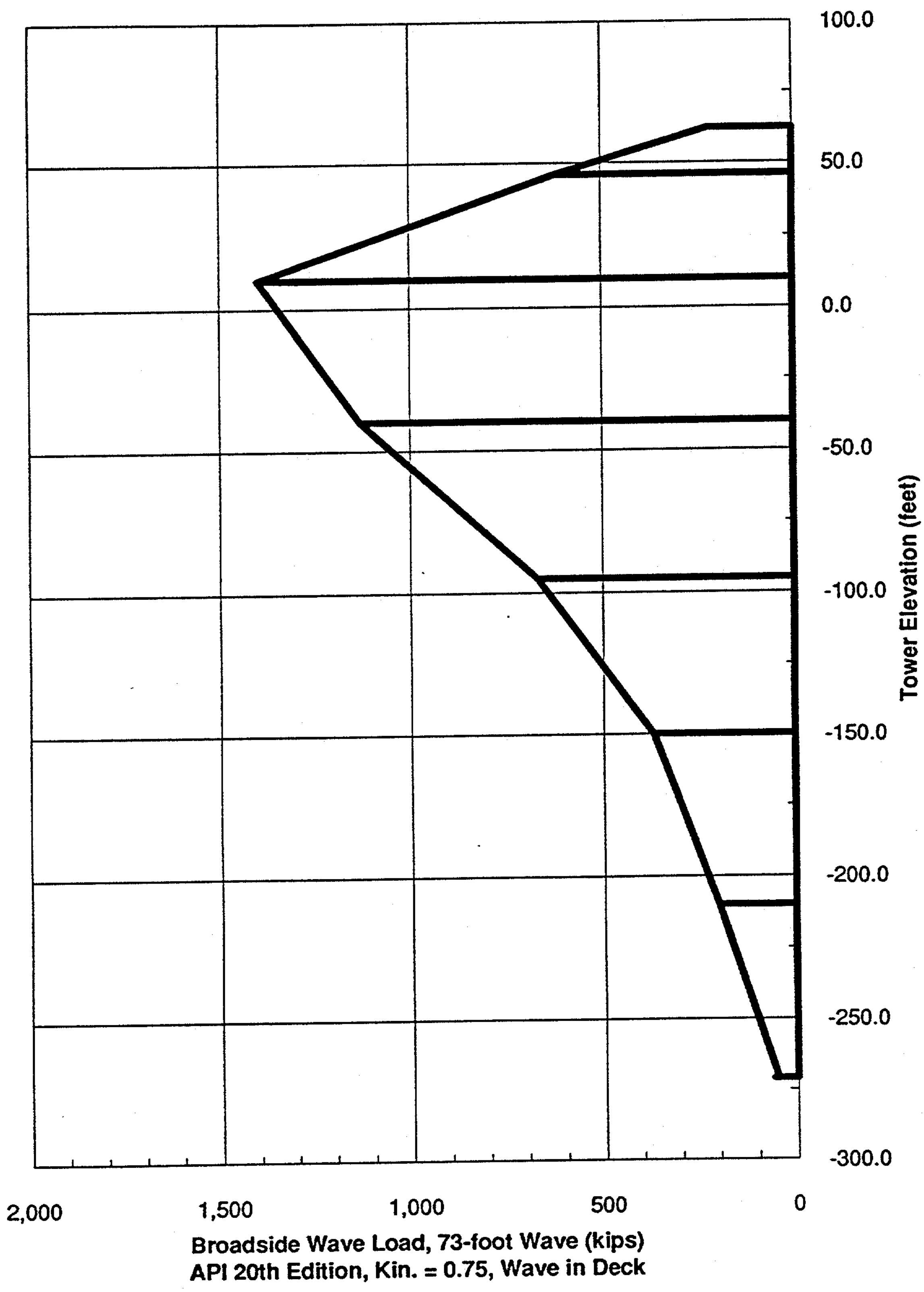
Figure 5-13 shows results of the static pushover for the two-dimensional Platform C computer model, base case condition, wave-in-deck, broadside direction. Figure 5-14 shows results for the three-dimensional model for the same conditions. The results are very similar with both models predicting a failure wave height of about 73-74 feet, equivalent to a base shear of about 4,900 kips for the three-dimensional model. Similar to the wave-below-deck condition, the pushover analysis was iterated several times until the difference between the pushover wave profile and the failure base shear were less than 5 percent.

Also shown on Figures 5-13 and 5-14 are the member failure sequences. The three-dimensional model collapses after braces in the jacket collapse. In the two-dimensional model, there is some yielding in the portal first, but structure collapse is the result of brace failure in the jacket, so the collapse processes of the two models are essentially the same.



WAVE HEIGHT VS BASE SHEAR

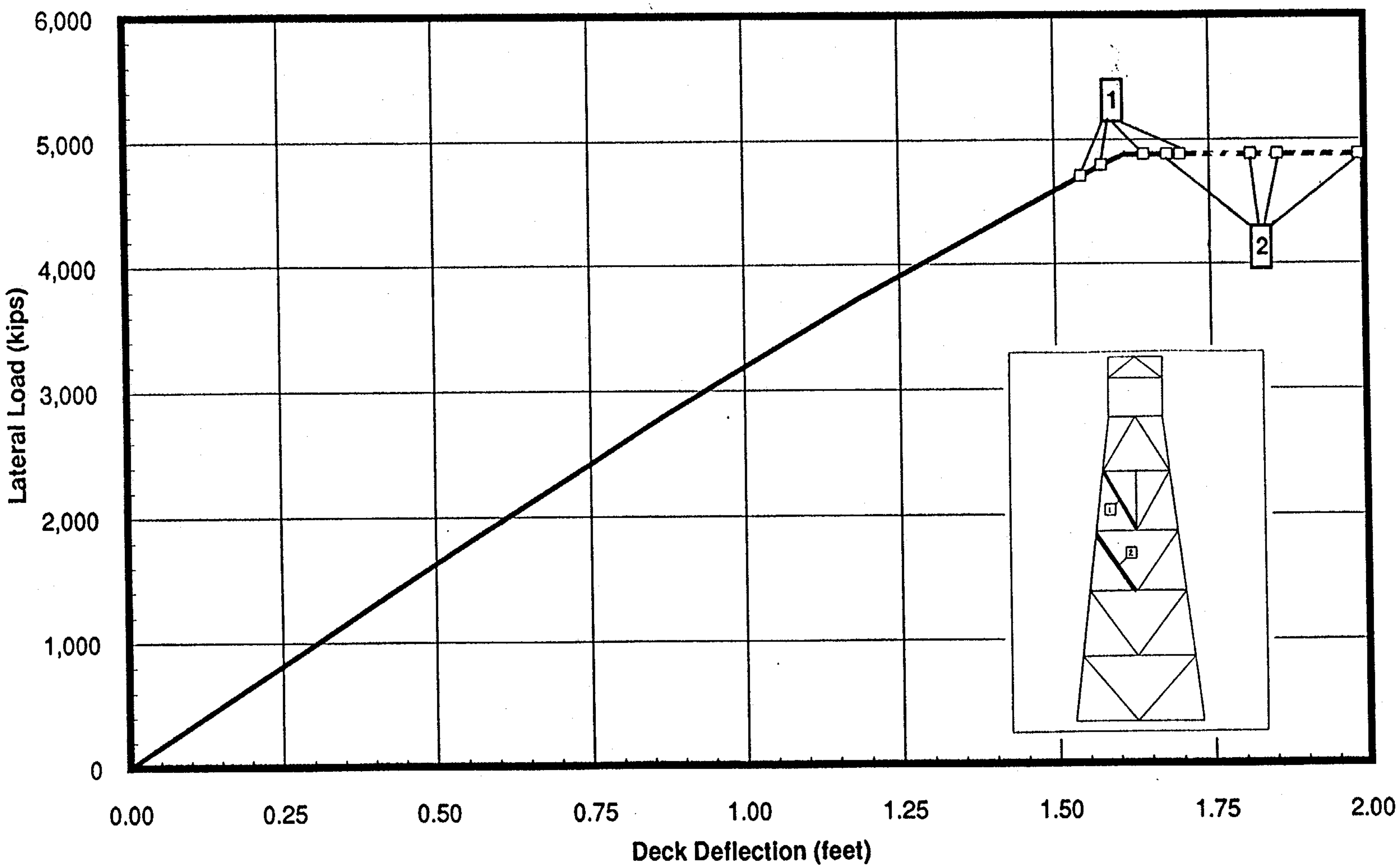
Fig. 5-1



PUSHOVER LOAD PROFILE

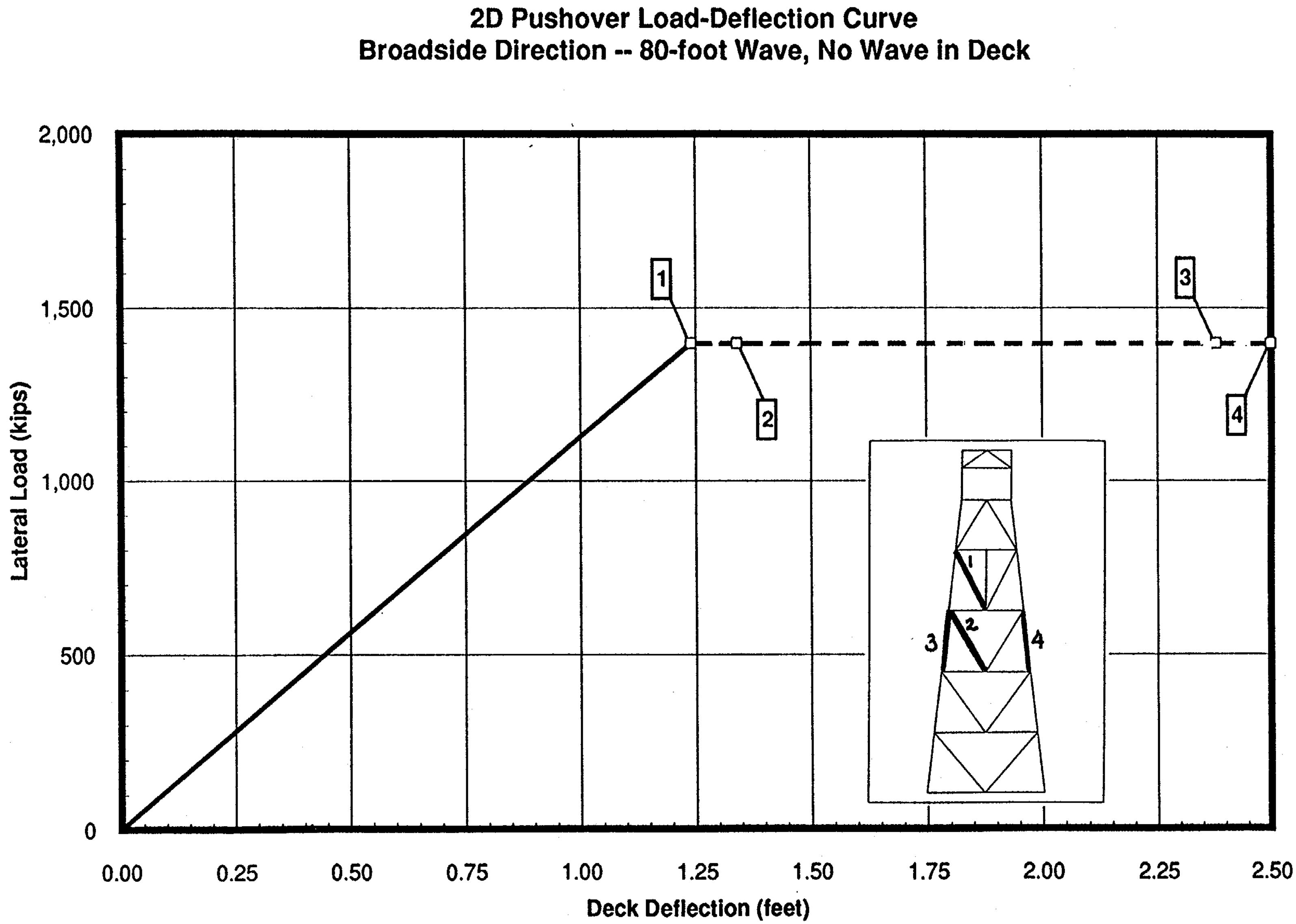
Fig. 5-2

3D Pushover Load-Deflection Curve
Broadside Direction -- 73-foot Wave, Wave in Deck



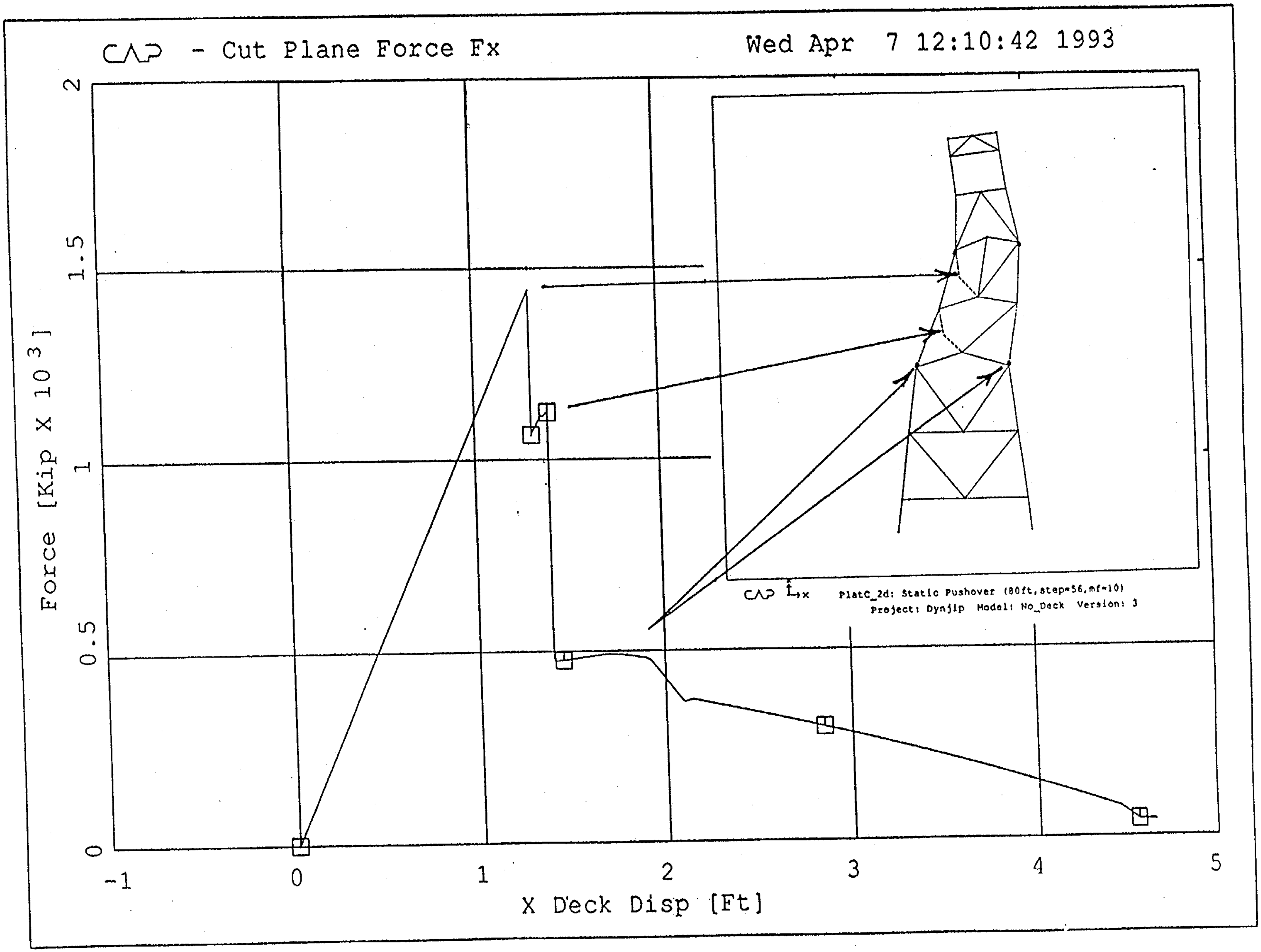
PUSHOVER RESULTS

Fig. 5-3



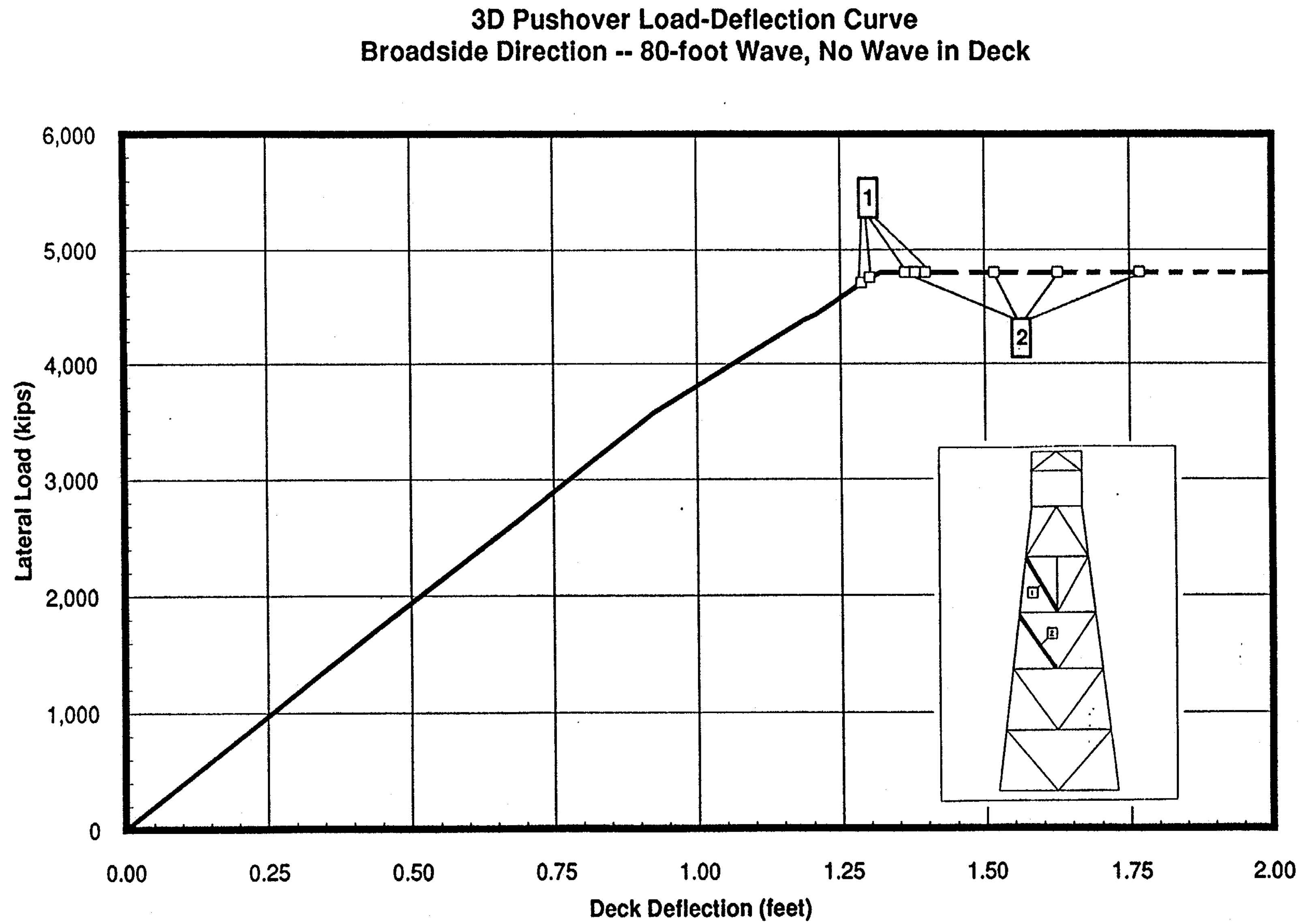
PSEUDO-STATIC PUSHOVER RESULTS

Fig. 5-4



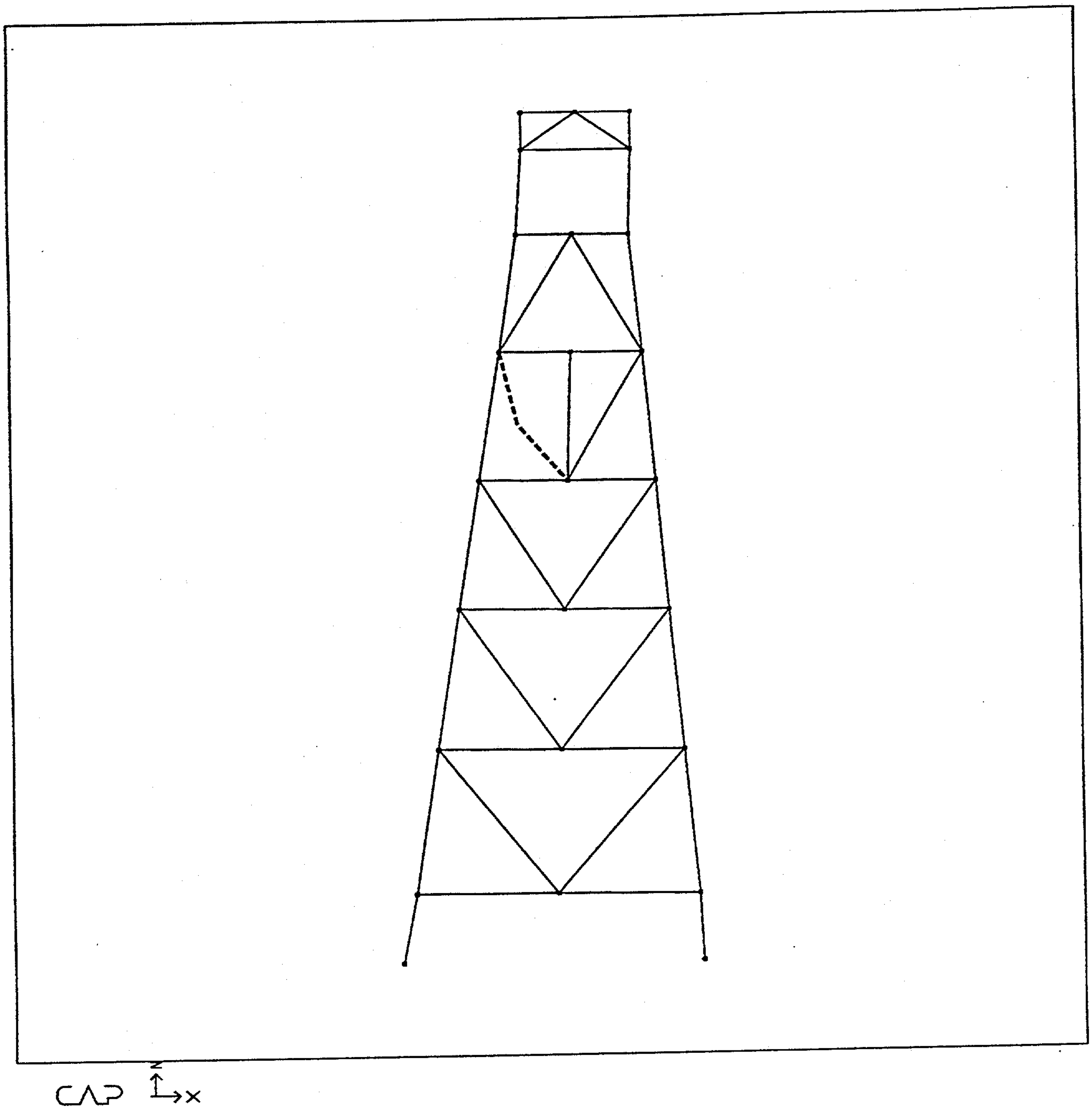
STATIC PUSHOVER RESULTS

Fig. 5-5



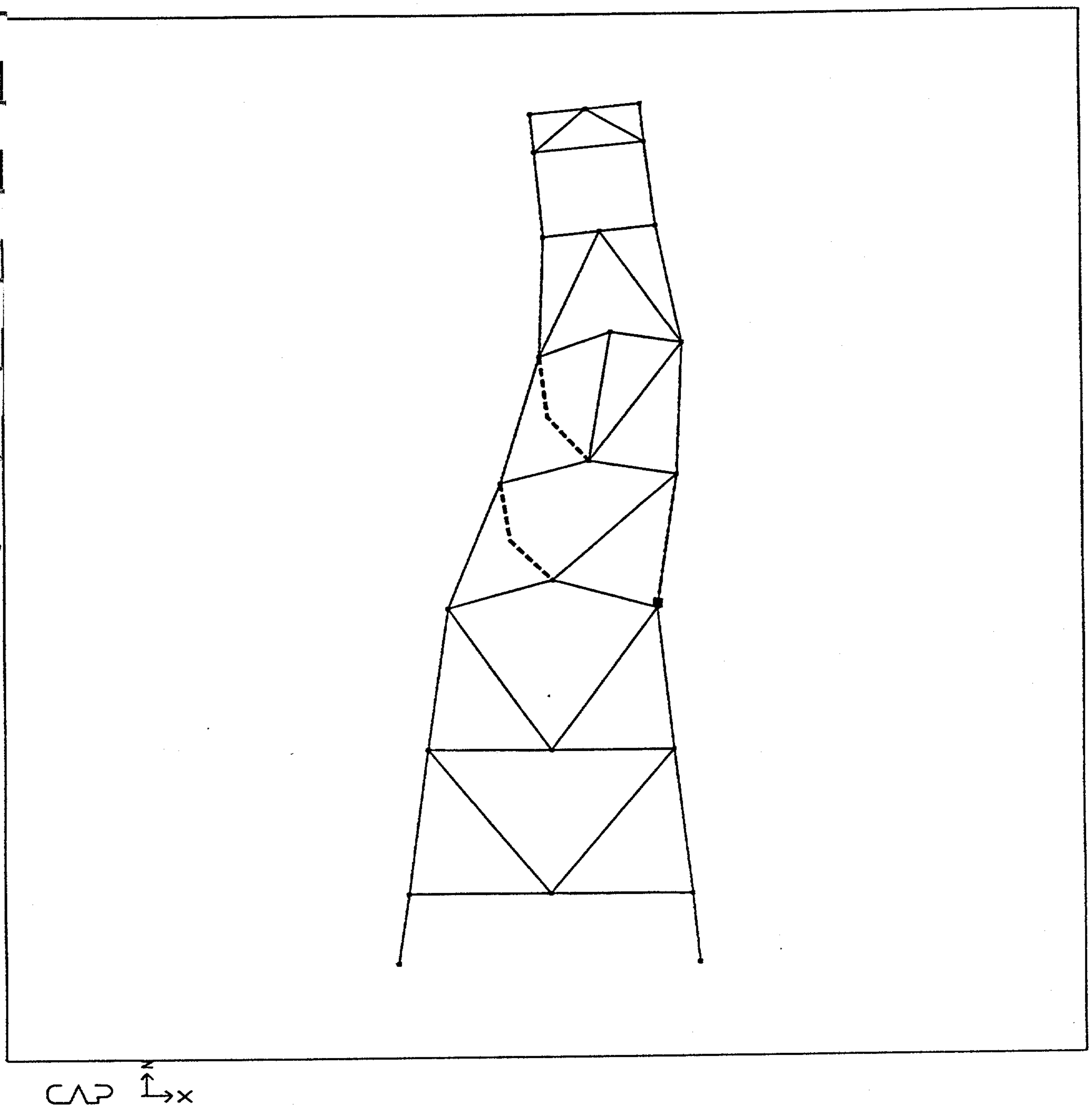
PUSHOVER. WAVE BELOW DECK

Fig. 5-6



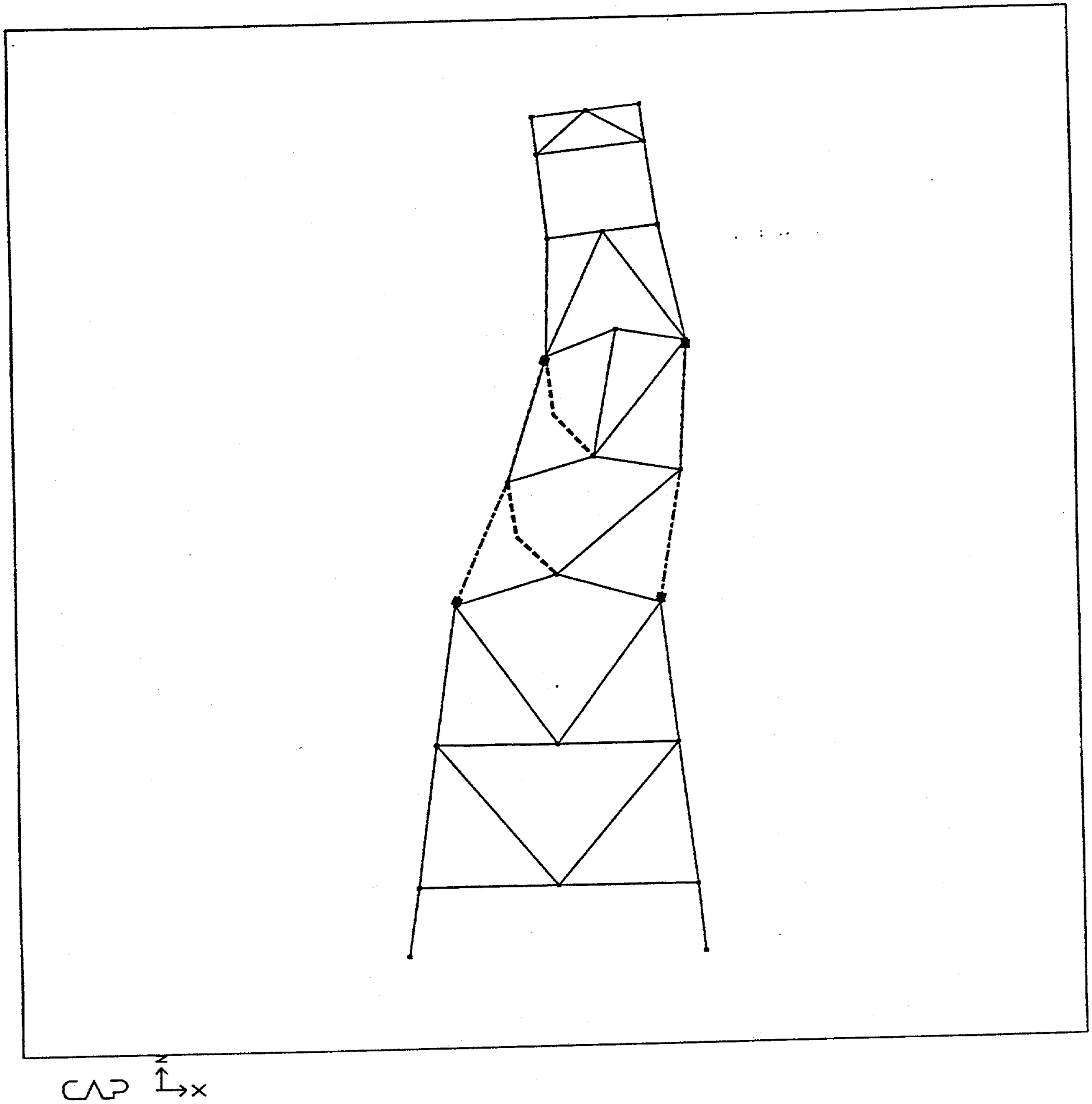
2-D. WAVE BELOW DECK. FIRST BRACE FAILURE

Fig. 5-7



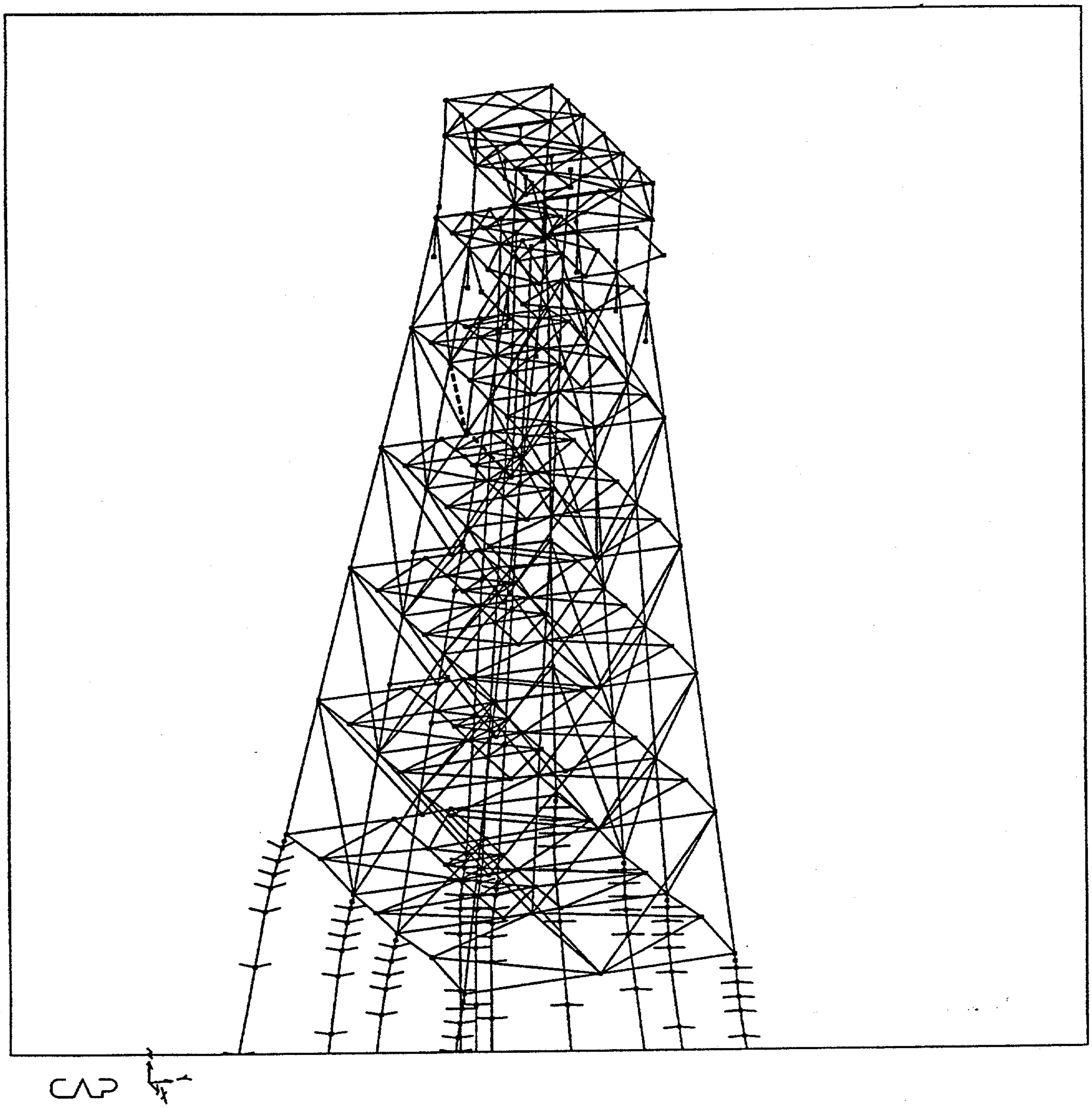
2-D. WAVE BELOW DECK. MULTIPLE BRACE FAILURES.

Fig. 5-8



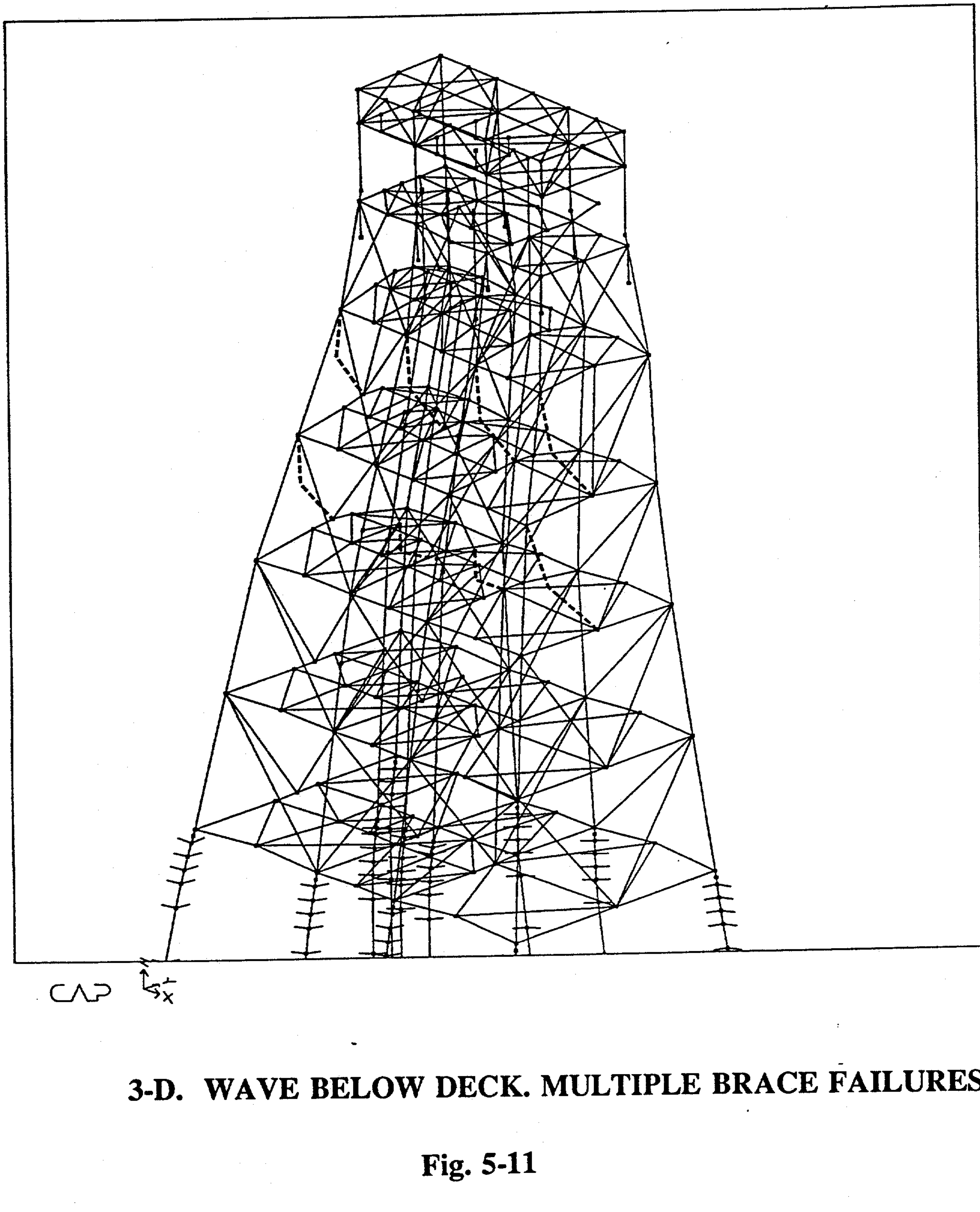
2-D. MULTIPLPE BRACE FAILURES
& LEG HINGING AT COLLAPSE

Fig. 5-9



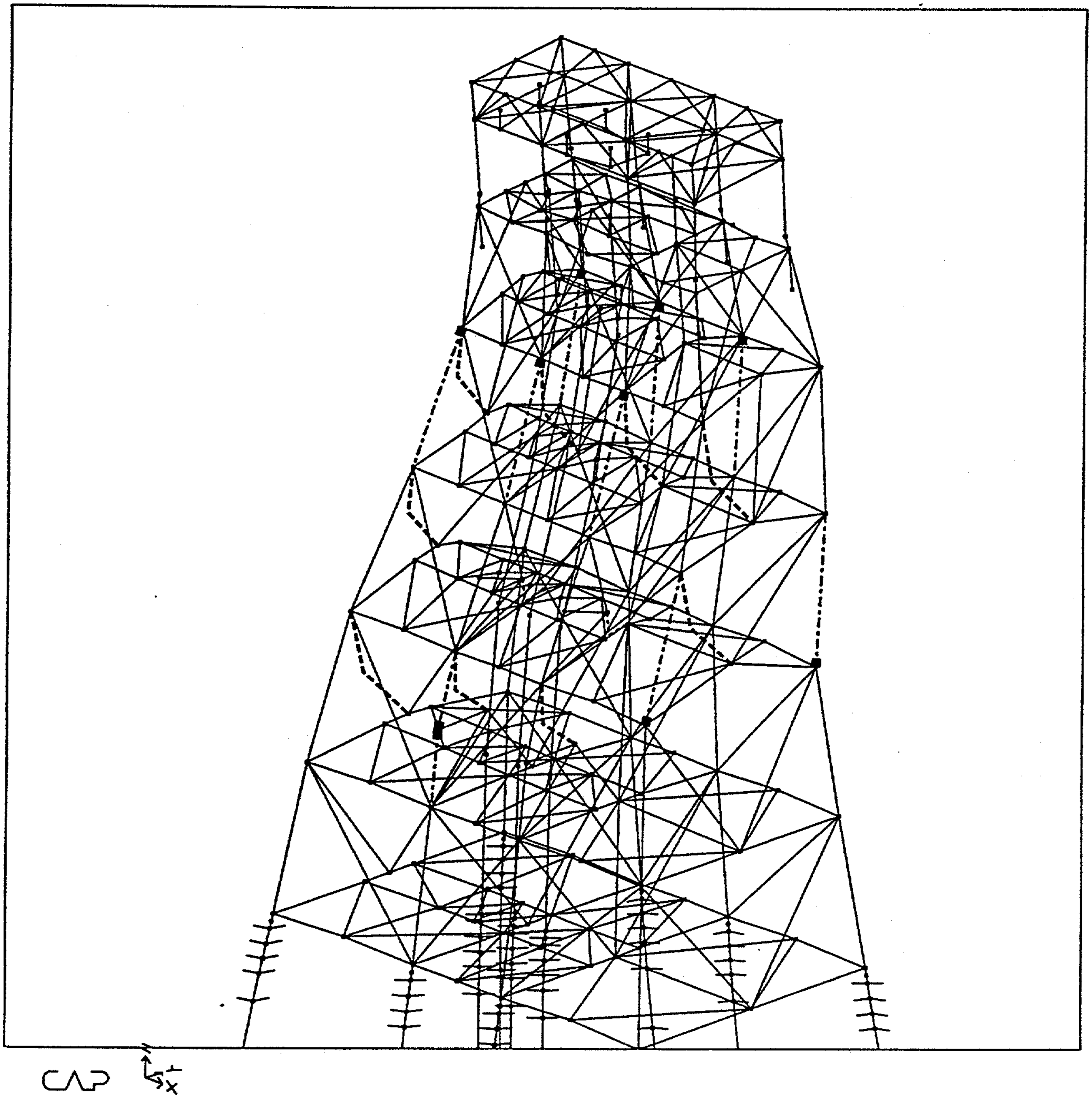
3-D. WAVE BELOW DECK. FIRST BRACE FAILURE.

Fig. 5-10



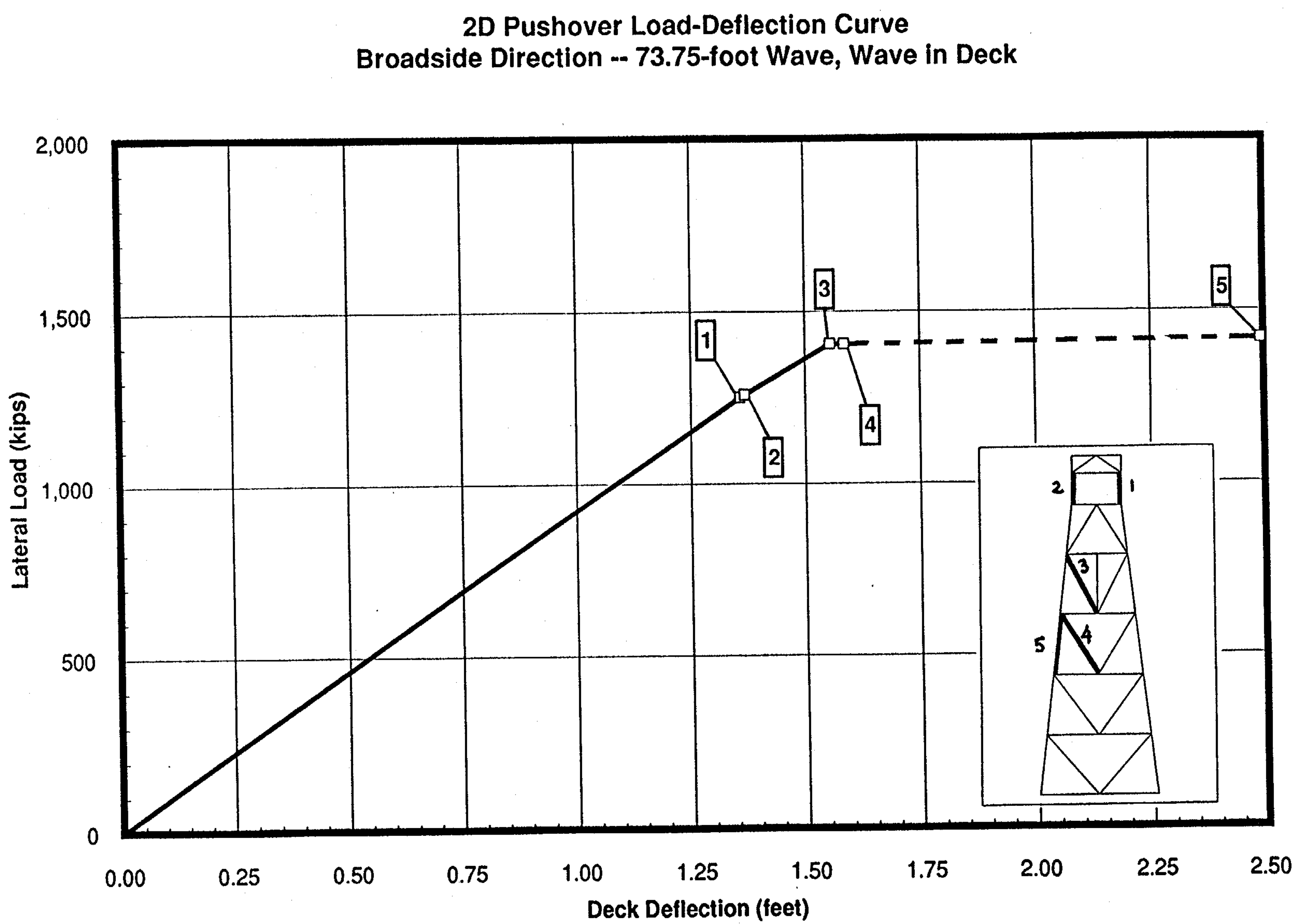
3-D. WAVE BELOW DECK. MULTIPLE BRACE FAILURES.

Fig. 5-11

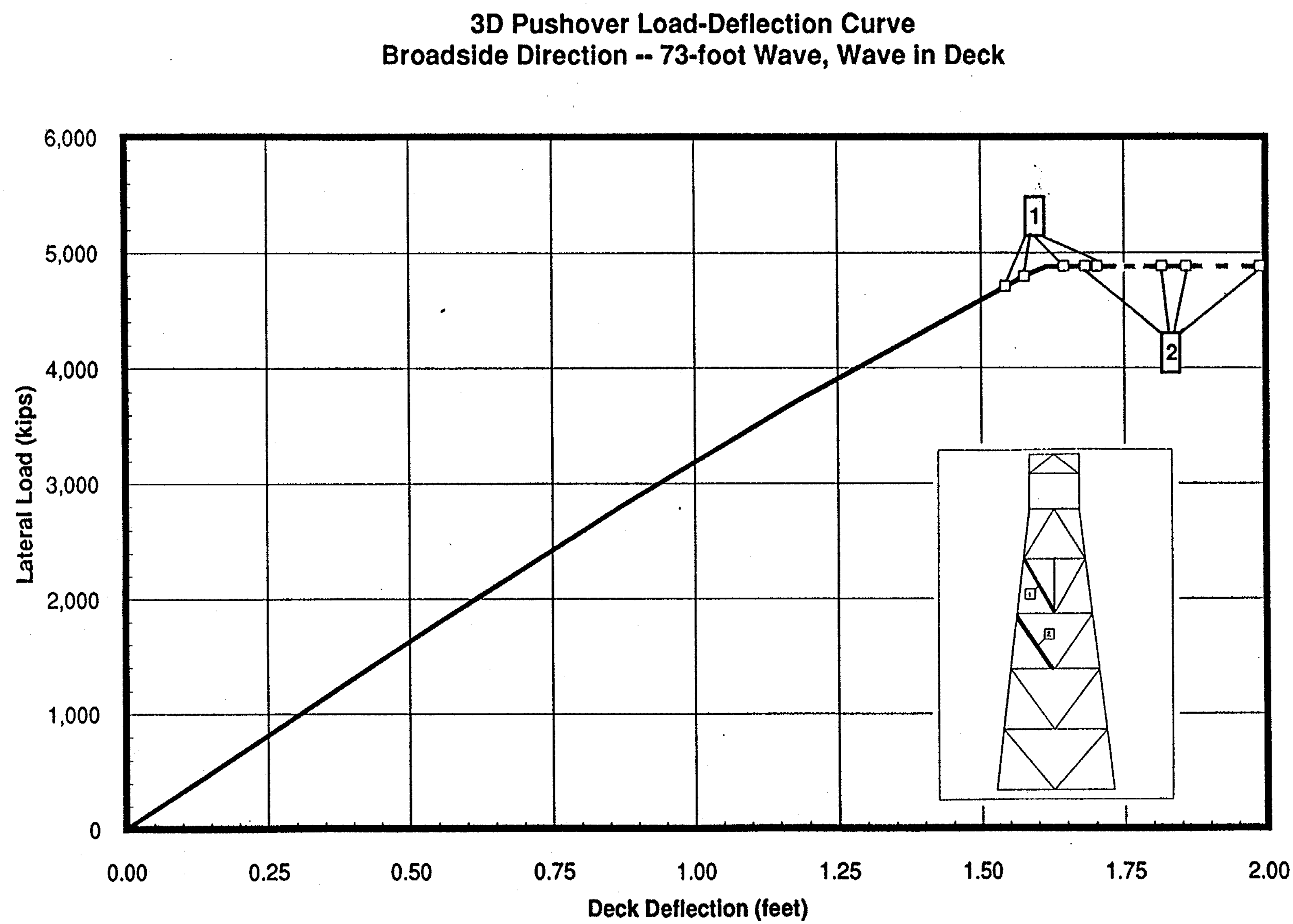


3-D. WAVE BELOW DECK. MULTIPLE BRACE FAILURES AND LEG HINGING AT COLLAPSE

Fig. 5-12



2-D PUSHOVER. WAVE IN THE DECK
Fig. 5-13



3-D PUSHOVER. WAVE IN THE DECK
Fig. 5-14

Section 6
Regular Wave Dynamic Capacity

6.1 APPROACH

The main objective of the project was to see whether there was any significant difference between the wave heights that caused the structure to collapse under static and dynamic conditions. This section describes studies on the models of Platform C under wave loads from regular dynamic waves.

The models used for the dynamic analyses were the same as those for the static pushover except that masses and damping, which were ignored in the static pushover, were included in these models. Both three-dimensional and two-dimensional models were studied with and without loads arising from waves and current in the deck.

The structure was loaded with deck loads, self weight, buoyancy, current and wind, all being applied as static loads. Then the wave loads were applied, with the analysis commencing near a trough of the wave in order to reduce transients. The previously applied static loads remained in effect. Two consecutive wave cycles of period 13.0 sec were included in the analyses, yielding 26.0 sec of simulation. The horizontal displacement histories of the deck were recorded along with the applied wave loads, and details of nonlinear events that took place in any members.

Waves somewhat smaller than the wave height that caused static collapse were tried first. A number of successive runs with approximately one foot increases in wave heights were then made, and for each the deck displacement history was examined. The last wave run had sufficient height that the deck displacement had increased greatly and the were starting to increase rapidly as the structure collapsed. This appeared to be a fairly sensitive definition of structure failure, as discussed in Section 6.4.

Results were collected after the analyses were complete, and were entered into summary tables such as Table 6-1.

6.2 TRANSIENTS

When a periodic load is applied to a structure which starts from rest with no initial displacements, it takes some time before the response settles down and starts to repeat itself, cycle after cycle. It was desired that the response would approximate the steady state response at the crests, and so care was taken to ensure that the effects of transients was not great.

The wave and current loads were quite low at the trough, the current approximately canceling out the wave loads. For this reason the wave loading was commenced at the trough. A test was made to see how significant the transients were under this loading. A wave of height 75.0 ft was run by the two-dimensional structure without computing deck wave loads. Five full cycles (65 sec) of simulation was used. Figure 6-1 shows the resulting deck displacement history. The transient, very pronounced at the beginning dies out over the five wave cycles, so that the last two deck displacement cycles closely repeat each other. The maximum displacement near the crest of the first wave was 1.04 times that of the steady state value of the last two waves. This results in a slight (4%) bias of all dynamic results. If more perfect initial conditions had been used, it would have required a wave load about 4% higher, to cause initial member failures. Assuming that wave forces vary as the square of wave heights, we therefore estimate, that in the dynamic studies following, the waves that cause failures would have been 2% lower, or about 1.5 ft lower for waves 60 - 80 ft high.

In some of the studies described in Section 8, the period of the structure increased considerably and there was some concern that the transient might be more important. As a check, a run was made in which the water particle kinematics were linearly ramped up from zero to full value over a three wave periods, after which two full wave loads were applied. This procedure greatly reduces transients and is described in Section 8.4.

6.3 DYNAMIC CAPACITY, TWO-DIMENSIONAL

The two-dimensional base case without deck wave loads is typical of all simulations and will be discussed in some detail.

For reference, the two-dimensional static pushover strength was 1390 kips from a 80.5 ft wave. This was addressed in Section 5.1.

Figure 6-2 shows the two-dimensional structure with the names of elements that will be used in these discussions.

The deck displacement history for a 77.0 ft wave is shown in Figure 6-3a. This wave produced no member nonlinear events, and except for a small effect from transients (See section 6.2) the response in the second wave cycle repeats the first. Figure 6-3b shows the total wave load history and the static pushover strength.

The 78.0 ft wave caused brace B3 to buckle near the crest of the first wave. (Figure 6-4). The structure did not collapse, however, since there was only a short time that the loading was near its peak. The structure was softened somewhat by the brace collapse, and it survived this and the next crest without collapse. Figure 6-5a shows the deck displacement history for this wave, and Figure 6-5b shows the wave load history.

When the wave height was increased to 79.0 ft, damage occurred at the first wave crest to the same brace member B3. With the increased load and length of time at this load, the compressive displacement in the member was ten times as much, so its strength after buckling was considerably lower, as it travelled along the buckling curve shown generically in Figure 4-9. At the next crest, brace B5 buckled (Figure 6-6) followed by partial yielding of the pile/leg composite section at various places (Figure 6-7). The resulting deck displacement at the second wave crest was almost five times that at the first (see Figure 6-8). The wave load history is shown in Figure 6-8b.

A further increase of the wave height to 80.0 ft gave the deck displacement history shown in Figure 6-9a. The structure is clearly at the verge of collapse even at the first crest, when braces B3 and B5 have buckled followed immediately by yielding in the legs L5 and L6. The structure does not actually fall over in the first crest, but at the second crest deformations increase, forming a collapse mechanism in the legs. The wave load history is shown in Figure 6-9b.

A well-defined collapse wave height is thus seen to exist for this structure. Depending on the deck displacement criterion used, the maximum wave height is seen to be either 79.0 or 80.0 ft. A wave height of 79.0 ft will be adopted here.

The applied wave load from wind, wave and current for the 79.0 ft wave was 1350 kips, which is very close to the capacity of 1390 kips determined by the static pushover.

For the 79 ft wave, the sequence of first nonlinear events in each member is also shown in Figure 6-8. The time of occurrence of events in each member is indicated with the deck displacement history.

Results were collected after the analyses were complete, and were entered into summary Table 6-1.

The first group of entries are the results of the static pushover analysis (Section 5.0), giving the wave height and associated total applied load, F_{stat}. This includes wind, wave and current. Failure mode is a brief description of the sequence of structural failure. P stands for portal, J for jacket. Thus P,J for instance, indicates that the portals were the first to fail but collapse included failure of braces and legs in the jacket.

The final group of entries are for the dynamic wave analyses.

The first group of data records information about the lowest wave height that caused any nonlinear events in the platform. Shown first are the wave height and the total applied load, F_{dyn}.

The final group of data shows information similar to that just described but for the wave height that caused collapse. This is defined as the wave height at which the displacements increase greatly (3 or 4 times) above those present before member failures commenced. Generally, this is followed by large displacements and inability of the analysis to reach convergence (without specifically adjusting the solution strategy, which was not considered important.) In order to compare the static collapse load directly with the dynamic collapse load, the ratio F_{dyn}/F_{stat} is also shown. If this ratio is 1.00 it means that the dynamic collapse load is the same as the static collapse load.

It can be seen from Table 6-1 that the static collapse load gives a very good estimate of the dynamic collapse load, with and without deck wave loads.

6.4 COLLAPSE DEFINITION

Defining structural collapse for static pushover is reasonably clear and essentially corresponds to the point when the vertical load carrying capability of the structure is lost. This undoubtedly corresponds to the formation of a portal mechanism in the main legs of the platform subsequent to the failure of the diagonal bracing system.

Definition of structural collapse is not as straightforward for dynamic, time domain analysis. The structure can be significantly damaged during the direct application of the large critical wave but due to the structure's inertia and the subsequent reversal of wave loading the structure may not develop enough lateral drift to create a collapse mechanism.

Various definitions of collapse were explored. Among these were:

1. **Local Deformation**, e.g. Story drift, Joint distortion, etc. Although the structure may not have actually fallen over, the damage to members or connections may be so great as to be considered intolerable.

2. **Residual Capacity**. The level of static pushover load that can be applied to the structure after it has been subjected to the dynamic wave crest could be used as a criterion of platform failure.

3. **Energy Criteria.** Inelastic energy can be monitored and when it exceeds some multiple of the elastic energy, structure failure could be considered to have occurred.

4. **Global Displacement.** Monitoring one or more points on the platform and setting some value above which the structure is considered to have failed. This was finally adopted for this project. At some wave height, the deck displacements started to

increase rapidly on the second wave crest, and this gave a rather precise definition of failure. Structure failure was deemed to have occurred when deck displacements had increased to four or more times the displacements before member yielding or failure had started.

From experience In applying these various measures, it is recommended that a combination of the Local Deformation and Global Displacement be used to provided the most consistent measure. Global displacement of the deck was the primary indicator of collapse level deformations. The vertical load carrying members (legs) would then be reviewed for portal hinge mechanisms.

6.5 DYNAMIC CAPACITY, THREE-DIMENSIONAL

The full three-dimensional model showed a similar dynamic response when subjected to regular waves. The following describes the response when subjected to regular wave without deck wave loads.

Due to the asymmetry of the position of the conductors, when loaded transversely, there is a considerable amount of torsion about a vertical axis. Figure 6-10 shows the displacement histories of two points on opposite sides of the deck, for a 75 ft wave, which is lower than causes any nonlinear events. Node 15 on the side closest to the conductors shows about a 20% higher peak displacement than node 13 on the other side.

When the wave height was increased to 78 ft, the jacket survived the first wave, but at the second crest the displacements were greater than 3 times those at the first, when the analysis failed to converge. The deck displacement history at the conductor side is shown in Figure 6-11, and this is superimposed with displacements on the other side in Figure 6-12. The first member failure in this wave occurred at the first crest, when the brace corresponding to B3 in the two-dimensional model, at the frame near the conductors, failed.

When the wave height was increased to 80 ft, more damage occurred at the first wave crest, but it still survived this crest, collapse not occurring until the second crest pushed the weakened jacket over. Figure 6-13 shows the deck displacement response at node 15.

Figures 6-14 through 6-17 show the three-dimensional structure at various stages during the 80 ft wave loading sequence. Figure 6-14 shows the first brace failure, corresponding in location to B3 in the two-dimensional model. (Figure 6-4). Figure 6-15 shows lower braces collapsing, followed by legs in Figure 6-17.

Table 6-1 includes a summary of the static and dynamic collapse of the three-dimensional structure when subjected to regular waves with and without deck wave loads. The entries are explained in Section 6.3. It can be seen that the static collapse load gives a very good estimate of the dynamic collapse load, with and without deck wave loads.

6.6 COMPARISON OF THREE-DIMENSIONAL AND TWO-DIMENSIONAL CAPACITIES

Tables 6-1 and 6-2 summarize the static and dynamic responses of the two-dimensional and three-dimensional models. The entries are explained in Section 6.3. Table 6-1 compares static and dynamic strengths of 2D and 3D models without deck loads. It can be seen that almost the same wave causes collapse in the 2D and 3D cases. With deck wave loads, although the deck loads were not the same for the 2D and 3D cases, it is seen from Table 6-2 that the collapse load, statically, is almost the same as that without deck wave loading. With this good agreement, it was considered appropriate to continue all further studies with the two-dimensional model.

6.7 COMMENTS ON DYNAMIC AMPLIFICATION FACTORS

With a sinusoidal loading of period 13.0 sec and a structure with first natural period of 2.1 sec, the dynamic amplification is 1.03 for damping 5% of critical. Thus the jacket wave load that could be expected to cause first member failure would be about 97% of the load that would cause first member failure when applied statically.

When the wave hits the deck, the time of loading is rather shorter than the wave period, and occurs only in one direction. The loading is very close to a half-period sine function as shown in Figure 6-18, for which dynamic amplification functions (DAF) can be derived. For waves just touching the deck, the duration t_1 of loading is essentially zero, so the DAF is also zero. When the wave is about 1 foot into the deck, it can be shown from the free surface history of the wave crest that the duration of loading is about 0.5 sec with a DAF of about 1.0. At 2 ft of deck inundation the duration is about 0.8 sec with a DAF of about 1.3. With 5 ft of wave in the deck the duration is about 1.2 sec, with DAF of 1.7.

It can be seen that the dynamic effects of wave in the deck are considerably more complicated than those in the jacket, since the DAF itself varies with the maximum depth of inundation of the deck.

For a linear structure it would be possible to estimate the peak load that the structure experiences for any wave. The effective jacket wave load contribution depends on the jacket peak wave load and the relative periods of the wave and the jacket natural period. The effective deck wave load depends on the peak deck wave load and the relative length of time of inundation and the jacket natural period.

But the calculation of collapse loads is another matter, since ductility characteristics determine what load can be carried after the first member failure. It is not likely that a calculation based on DAF's of the jacket and deck wave forces would, in general, yield an accurate estimate of the collapse load, since the collapse load depends on the ductility of the structure, or putting it another way, it depends on the changing periods of the structure as members fail sequentially.

For this reason it is expected that the only way to determine collapse loads analytically is through nonlinear analyses of the type used in this project, and not through attempting to scale up peak wave loads with DAF's, and apply these in a static analysis.

6.8 BRACE DYNAMIC CAPACITY STUDY

At the start of the project a brief study was made of the effects of dynamics on the axial capacity of struts. This study is now described.

A model was made of a typical platform brace. The brace was pinned at the ends and was flooded. The initial configuration of the brace included a sinusoidal out-of-straightness of 0.15% of its length.

The properties of the brace are as follows:

Diameter	42 in.
Thickness	3/4 in.
Length	148.5 ft
Marine Growth	3 in.

The periods of the first two modes were

Mode 1	1.36 sec
Mode 2	0.34 sec

The elastic buckling load was 1934 kips.

The strut was modeled with six nonlinear beam-columns elements in series that yield under a combination of bending and axial forces.

The static strength of the member was first established by applying axial compressive displacements, increasing in magnitude until the load started to fall off as the member collapsed at a peak load of 1640 kips. The resulting force-displacement relation is shown in Figure 6-19.

Next the member was loaded with an axial load that varied sinusoidally with the short period of 2.0 sec, from zero to a maximum value and back to zero again, while the displacement history was recorded. Figure 6-20 shows such a plot for a maximum load of 2650 kips. This test was then repeated with maximum axial loads of 2750 kips, 2850 kips, 2950 kips and 3050 kips.

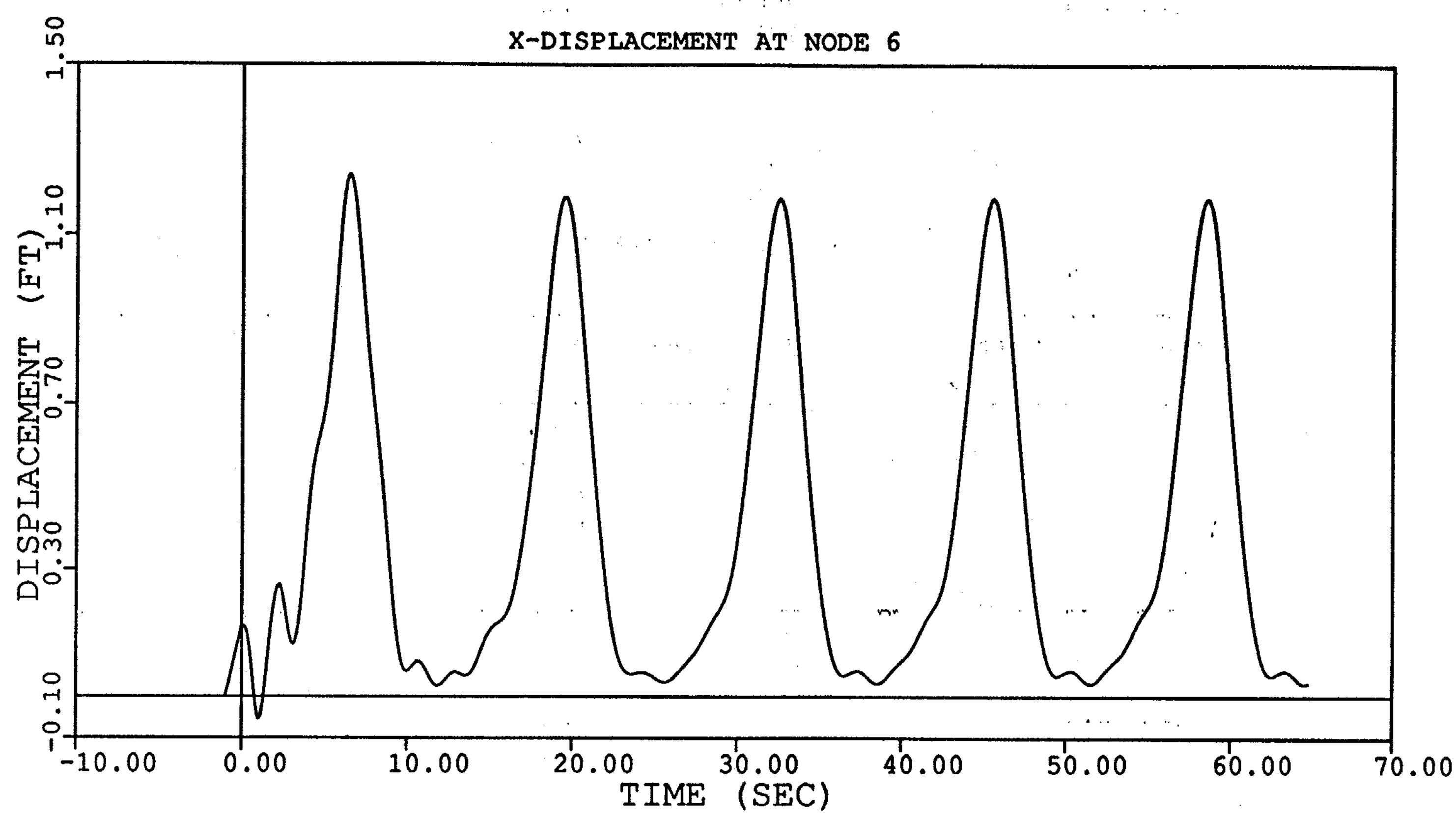
For the lowest maximum load tested, the loaded end returned to almost its initial position, as shown in Figure 6-20 and in Figure 6-21 showing the applied force against end displacement. As the maximum load increased the permanent deformation of the end at the end of the cycle increased, and at a maximum load of 2950 kips, the end did not return at all as the load was removed. (Figure 6-22). At 3050 kips the structure continued to shorten even as the load was removed. (Figures 6-23, 6-24). The collapse load was defined as the first load at which deflection continued to increase throughout the loading. This was estimated to be 3000 kips for this loading. This is considerably larger than the static collapse load of 1640 kips, indicating considerable increase in strength resulting from the effect of dynamics in the strut.

The test was repeated with a period of 12.0 seconds to represent typical wave periods. A maximum load of 1650 kips showed almost complete recovery when the load was removed (Figure 6-25), but when this was increased to 1700 kips, the structure continued to shorten axially until convergence of the solution failed (Figure 6-26). The collapse load was estimated as 1675 kips. This is only slightly higher than the static collapse load. Thus for this member, the strength is hardly affected by dynamic effects, when the applied load has a period of 12.0 sec.

Table 6-3 summarizes results.

The conclusion from this study is that the axial strength of the member is hardly affected when the applied load period is 12.0 sec, at typical wave period, while it is almost doubled when the period is 2.0 sec. Thus for wave loading it is not expected that the dynamics of braces will lead to any significant change in strength.

PMB Engineering Inc., San Francisco - Houston

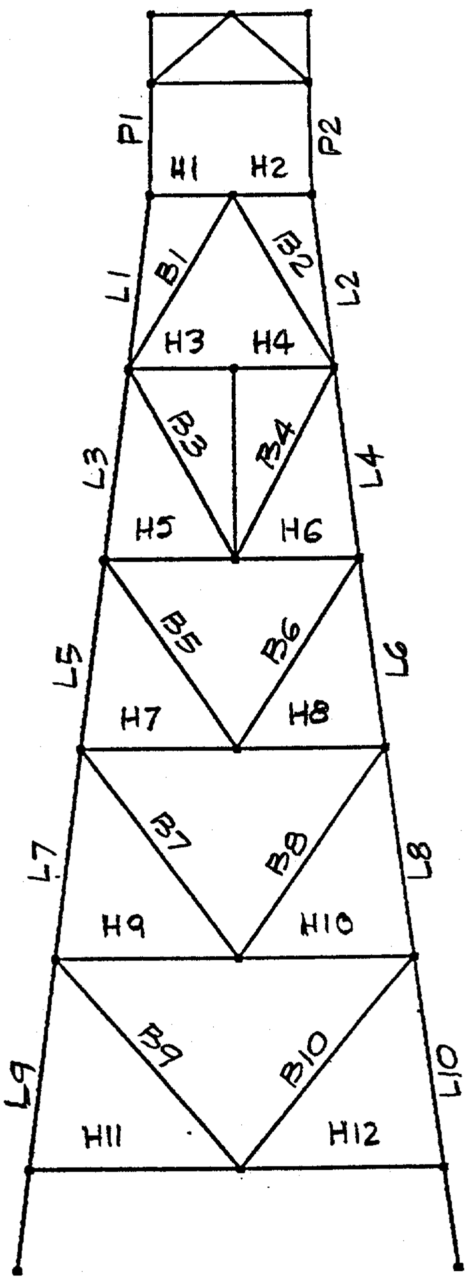


2-D MODEL. CHECK OF TRANSIENTS. 79 FT WAVE

DISPLACEMENTS AT DECK

DATE - 07/29/93 SEAPOST Version 3.10 TIME - 13:30:17

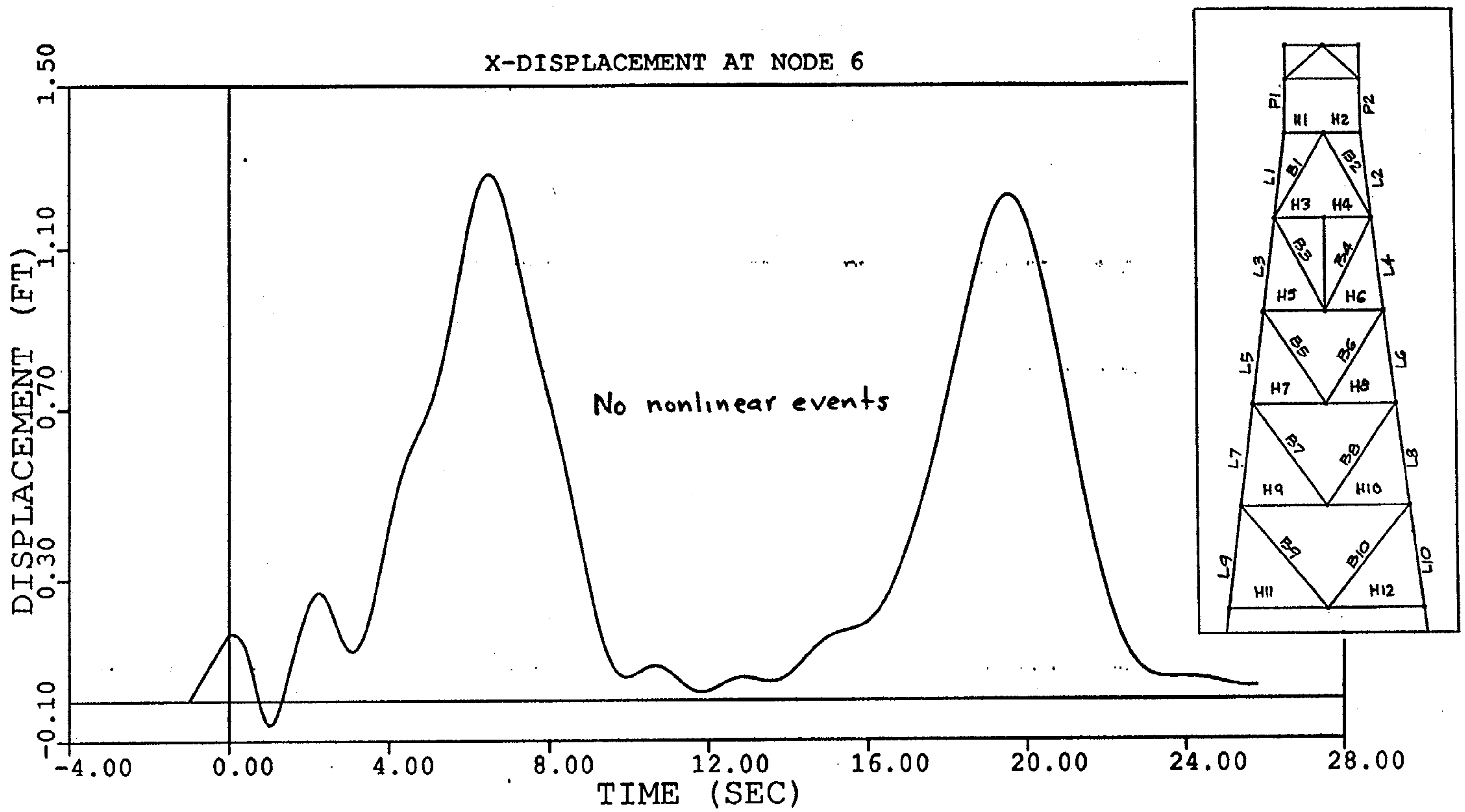
Fig. 6-1



2-D Model Member Names for Failure Sequence Descriptions

Fig. 6-2

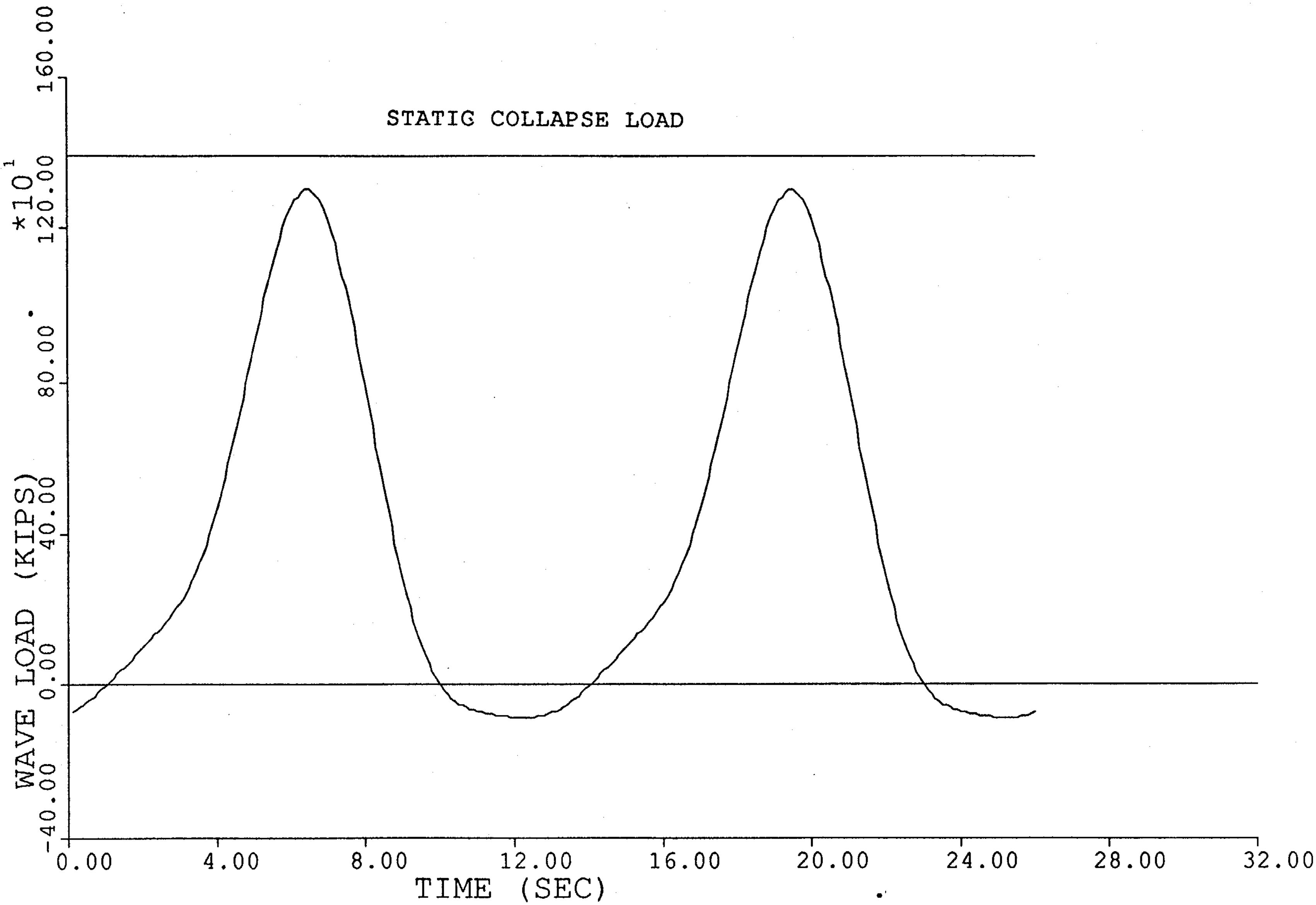
PMB Engineering Inc., San Francisco - Houston



2-D MODEL. 77 FT WAVE. NO WAVE IN DECK
DISPLACEMENTS AT DECK

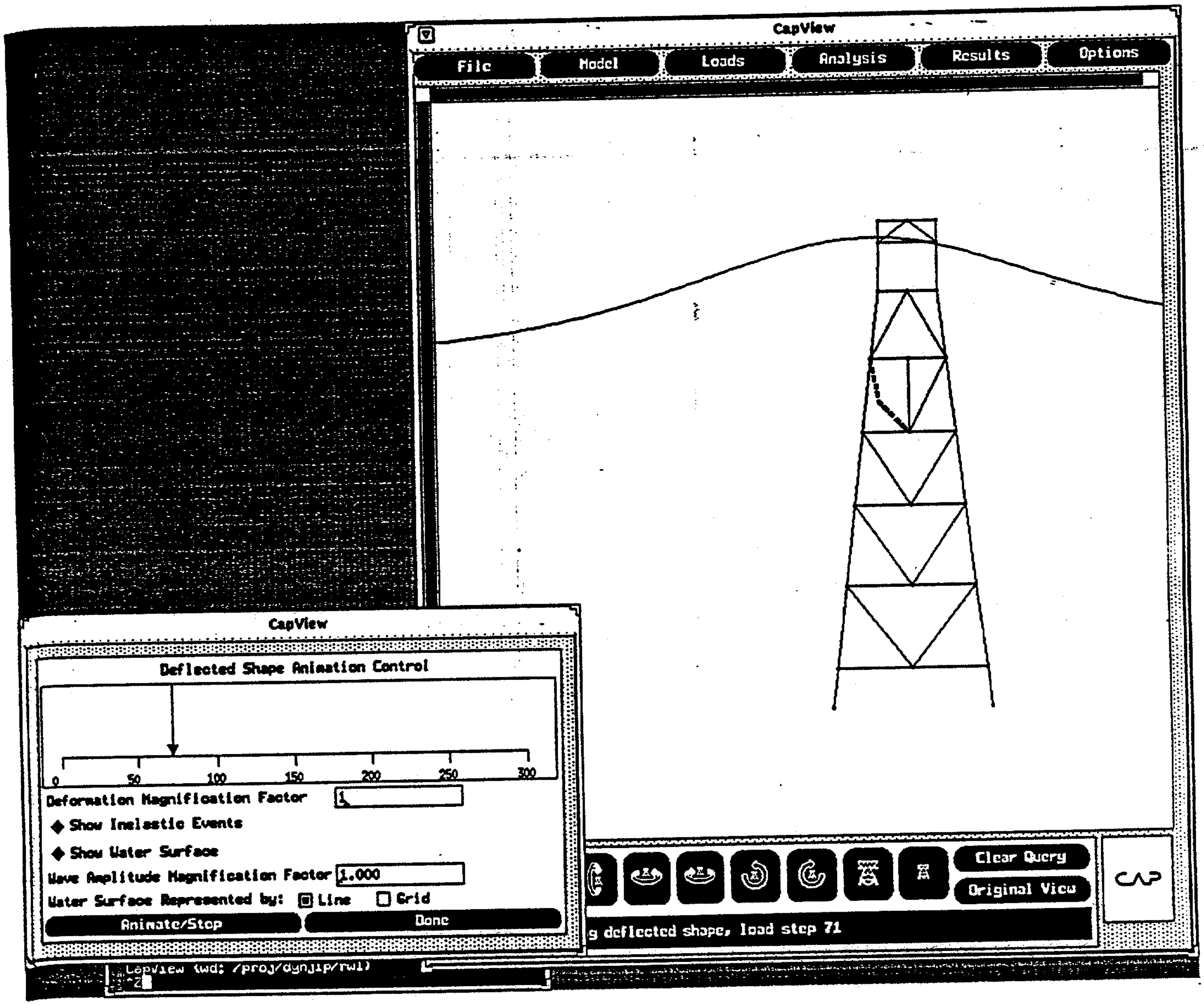
DATE - 07/29/93 SEAPOST Version 3.10 TIME - 07:56:42

Fig. 6-3a



2-D. 77 FT. NO WAVE IN DECK. WAVE, WIND LOAD

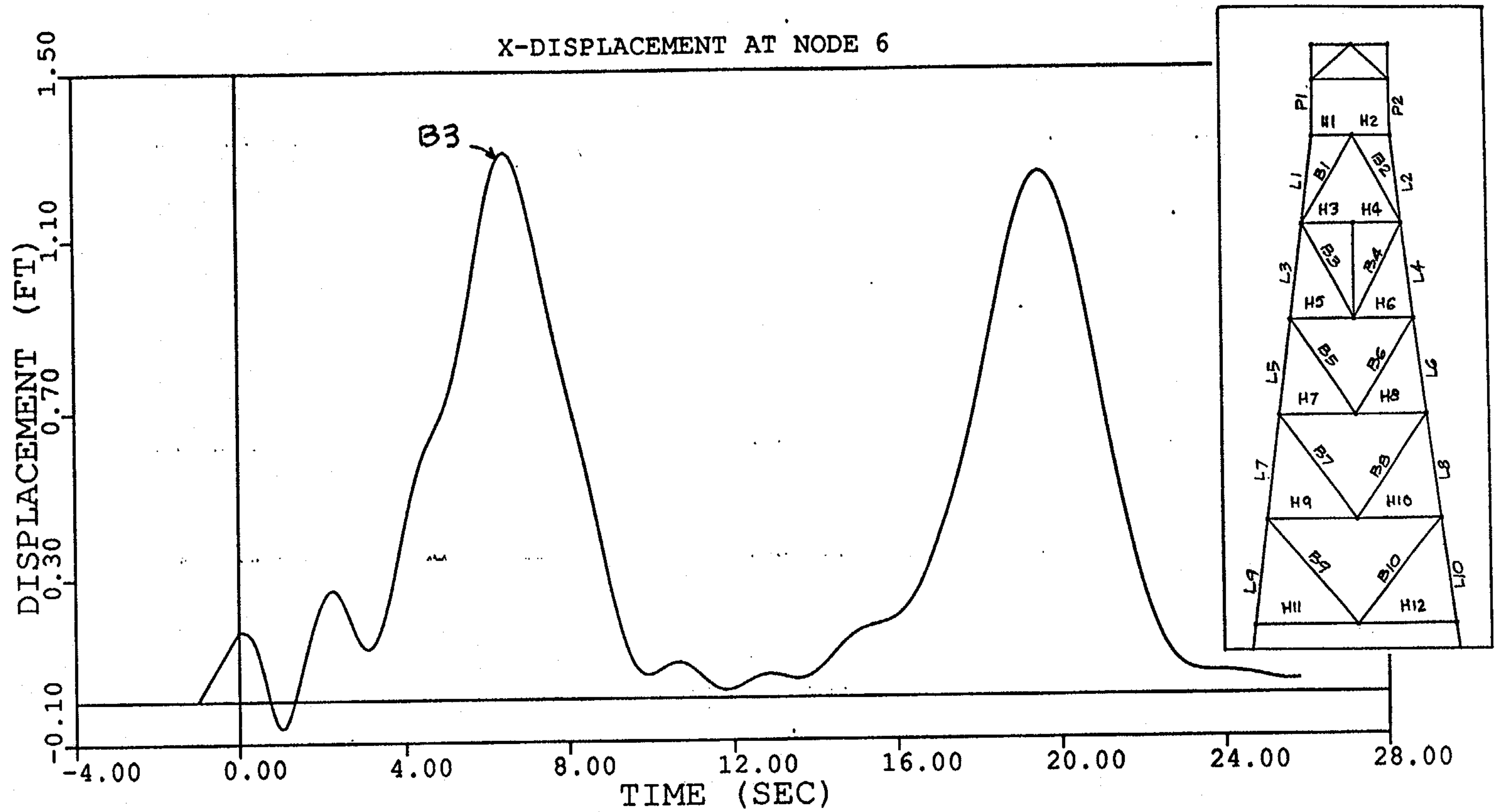
Fig. 6-3b



2-D MODEL. NO WAVE IN DECK. 78 FT WAVE

Fig. 6-4

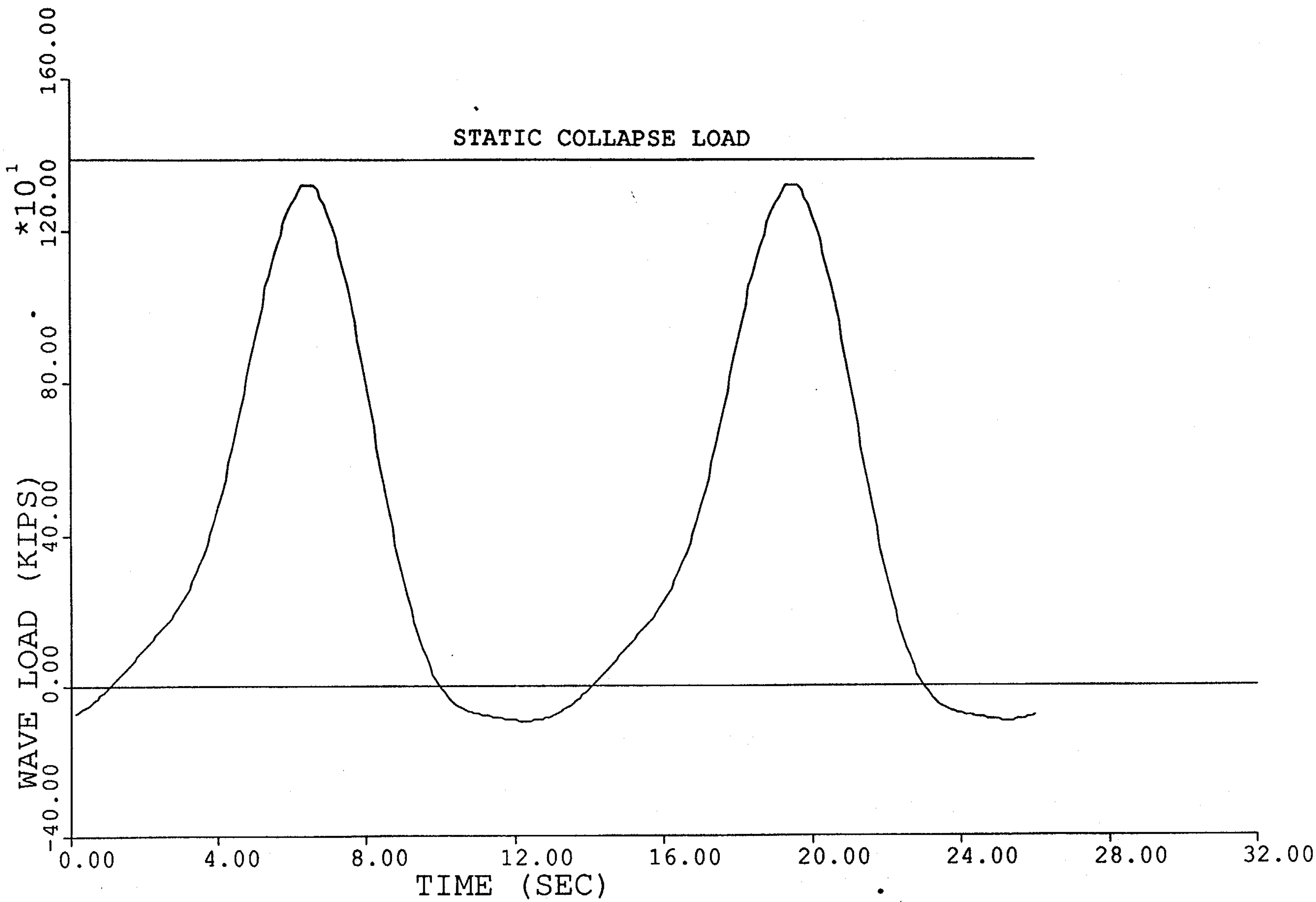
PMB Engineering Inc., San Francisco - Houston



2-D MODEL. 78 FT WAVE. NO WAVE IN DECK
DISPLACEMENTS AT DECK

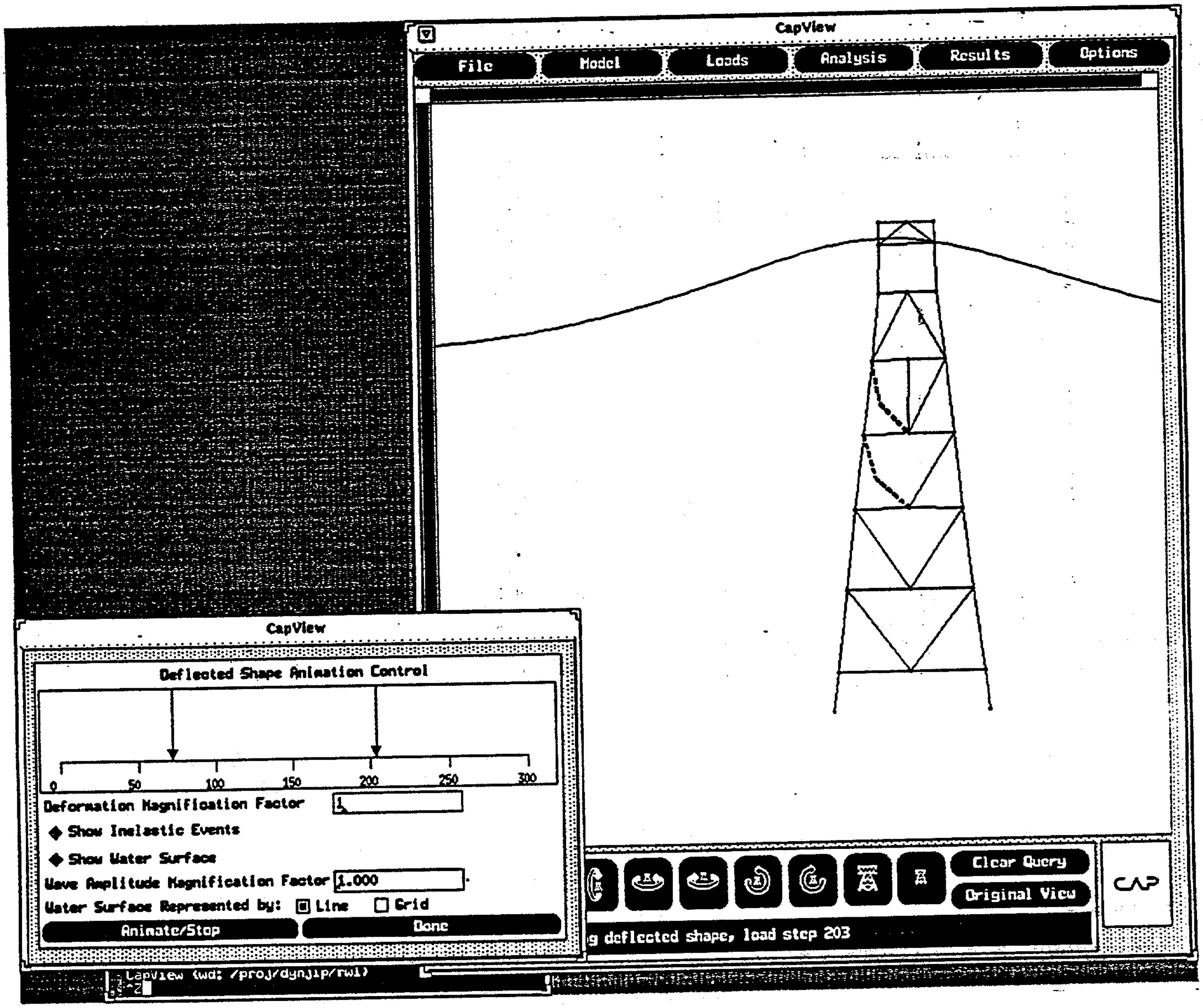
DATE - 07/29/93 SEAPOST Version 3.10 TIME - 08:11:17

Fig. 6-5a



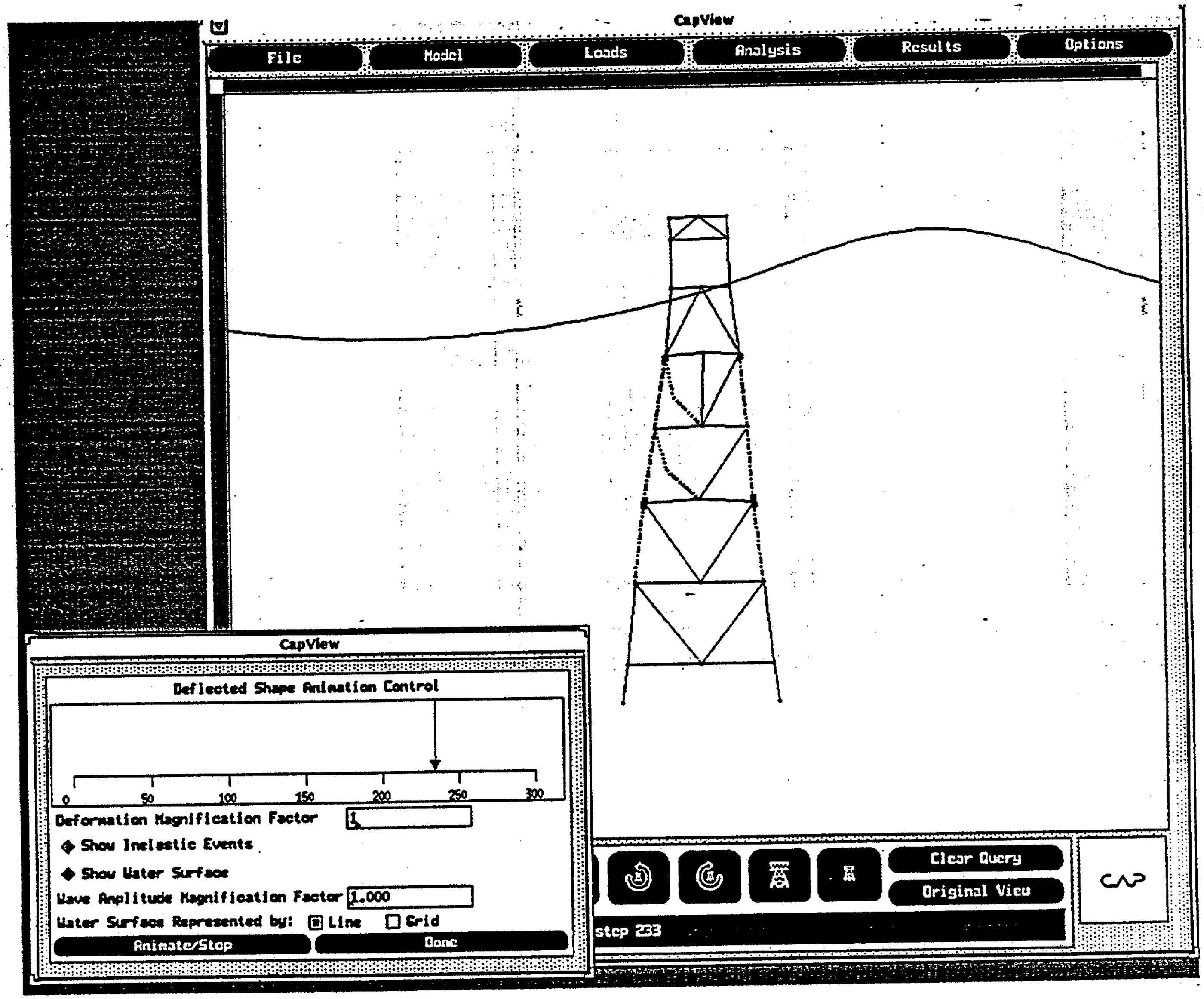
2-D. 78 FT. NO WAVE IN DECK. WAVE, WIND LOAD

Fig. 6-5b



2-D MODEL. NO WAVE IN DECK. 79 FT WAVE

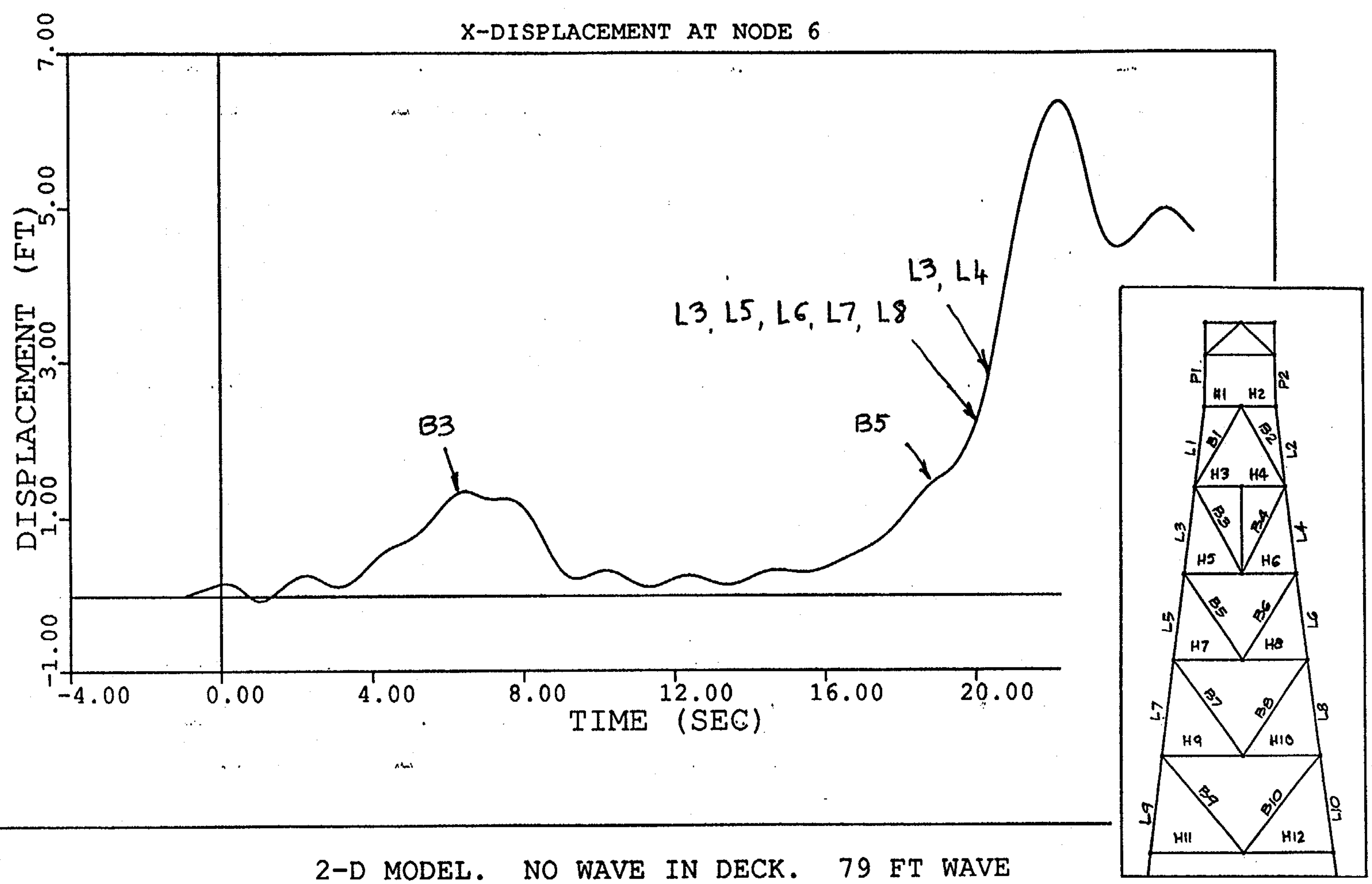
Fig. 6-6



2-D MODEL. NO WAVE IN DECK. 79 FT WAVE

Fig. 6-7

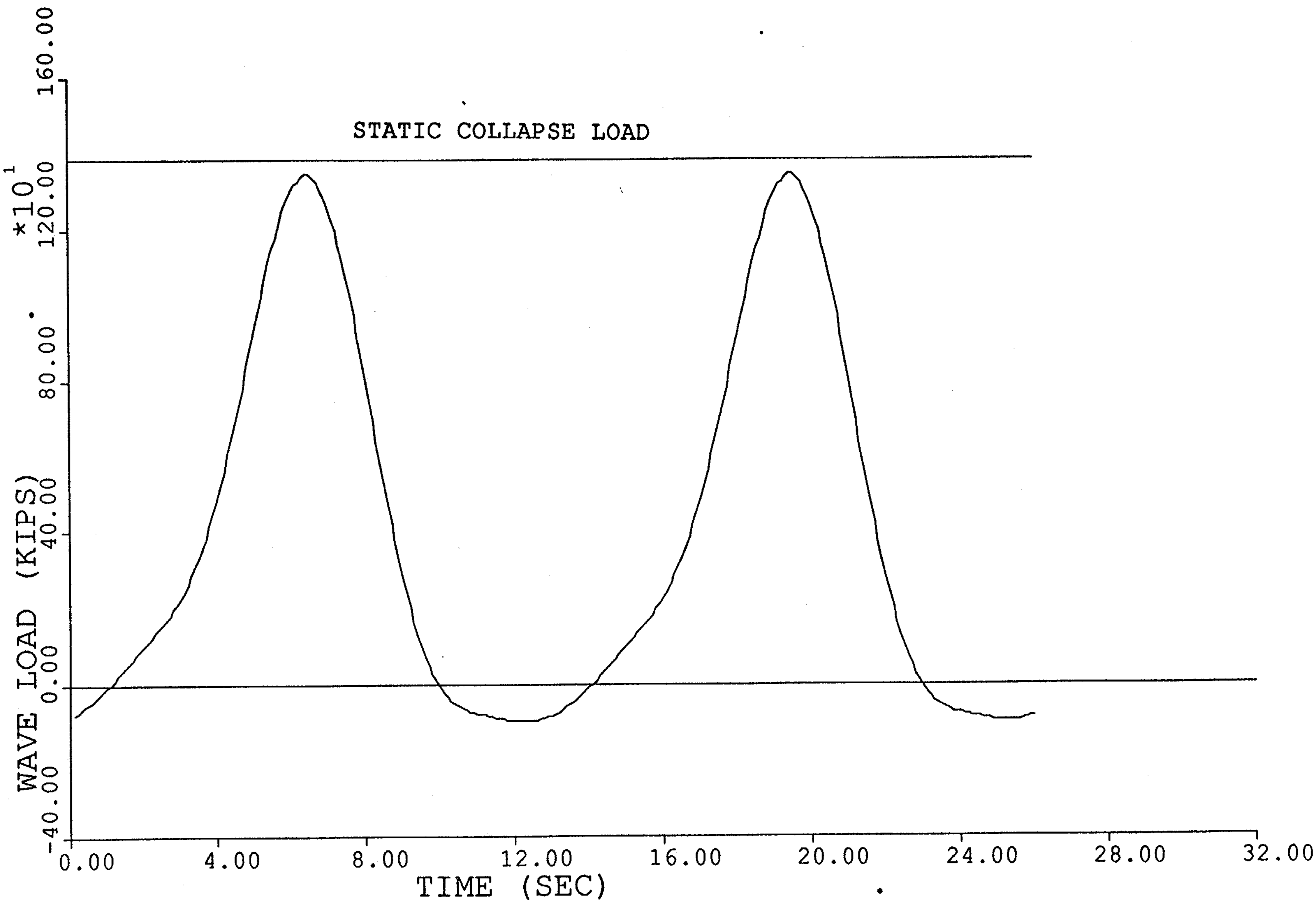
PMB Engineering Inc., San Francisco - Houston



2-D MODEL. NO WAVE IN DECK. 79 FT WAVE
DISPLACEMENTS AT DECK

DATE - 07/29/93 SEAPOST Version 3.10 TIME - 08:26:11

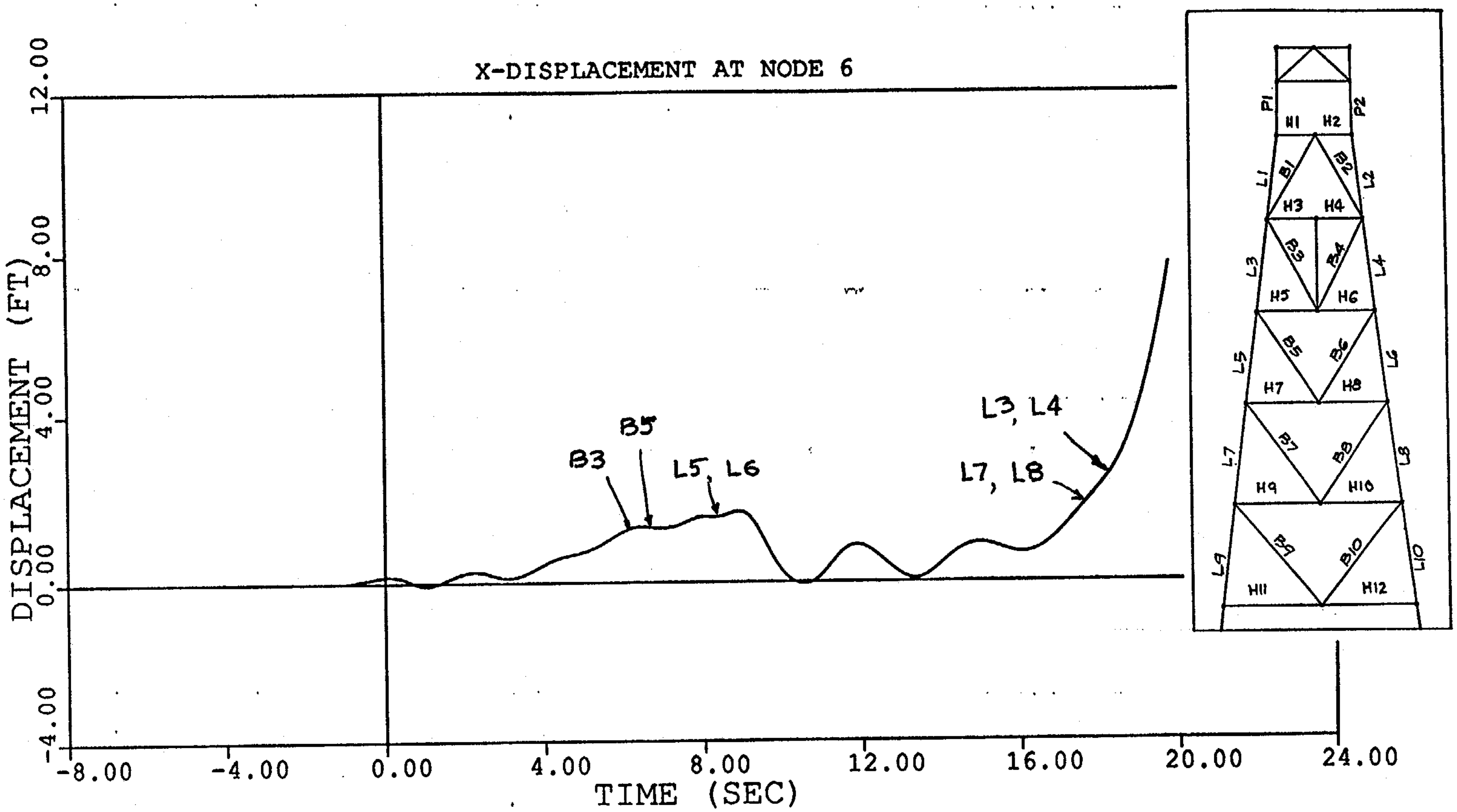
Fig. 6-8a



2-D. 79 FT. NO WAVE IN DECK. WAVE, WIND LOAD

Fig. 6-8b

PMB Engineering Inc., San Francisco - Houston

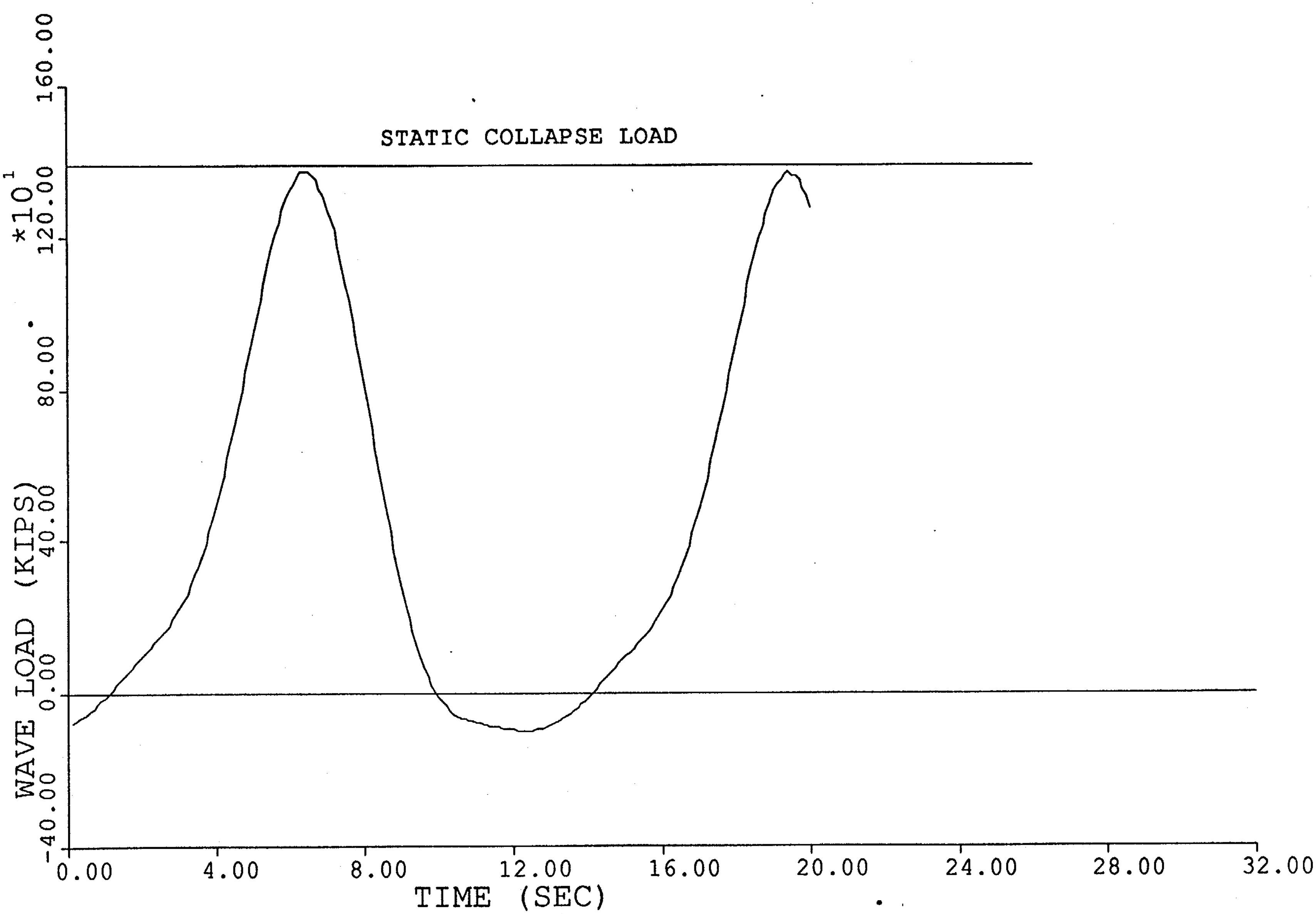


2-D MODEL. NO WAVE IN DECK. 80 FT WAVE

DISPLACEMENTS AT DECK

DATE - 07/29/93 SEAPOST Version 3.10 TIME - 11:33:37

Fig. 6-9a



2-D. 80 FT. NO WAVE IN DECK. WAVE, WIND LOAD

Fig. 6-9b

3-D MODEL. NO WAVE IN DECK. 75 FT WAVE

Deck Displacement

(NO MEMBER FAILURES)

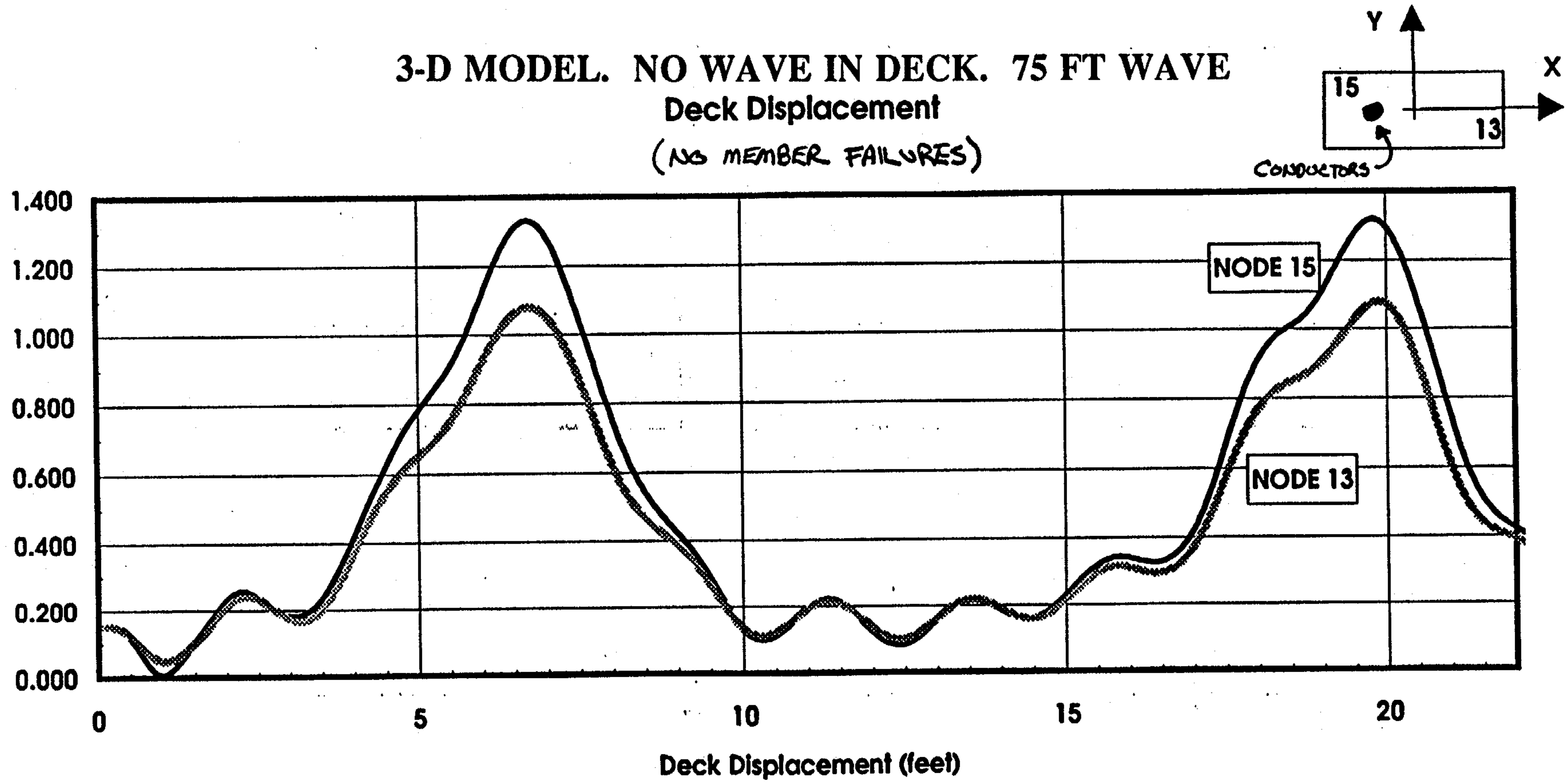


Fig. 6-10

3-D MODEL. NO WAVE IN DECK. 78 FT WAVE

Deck Displacement

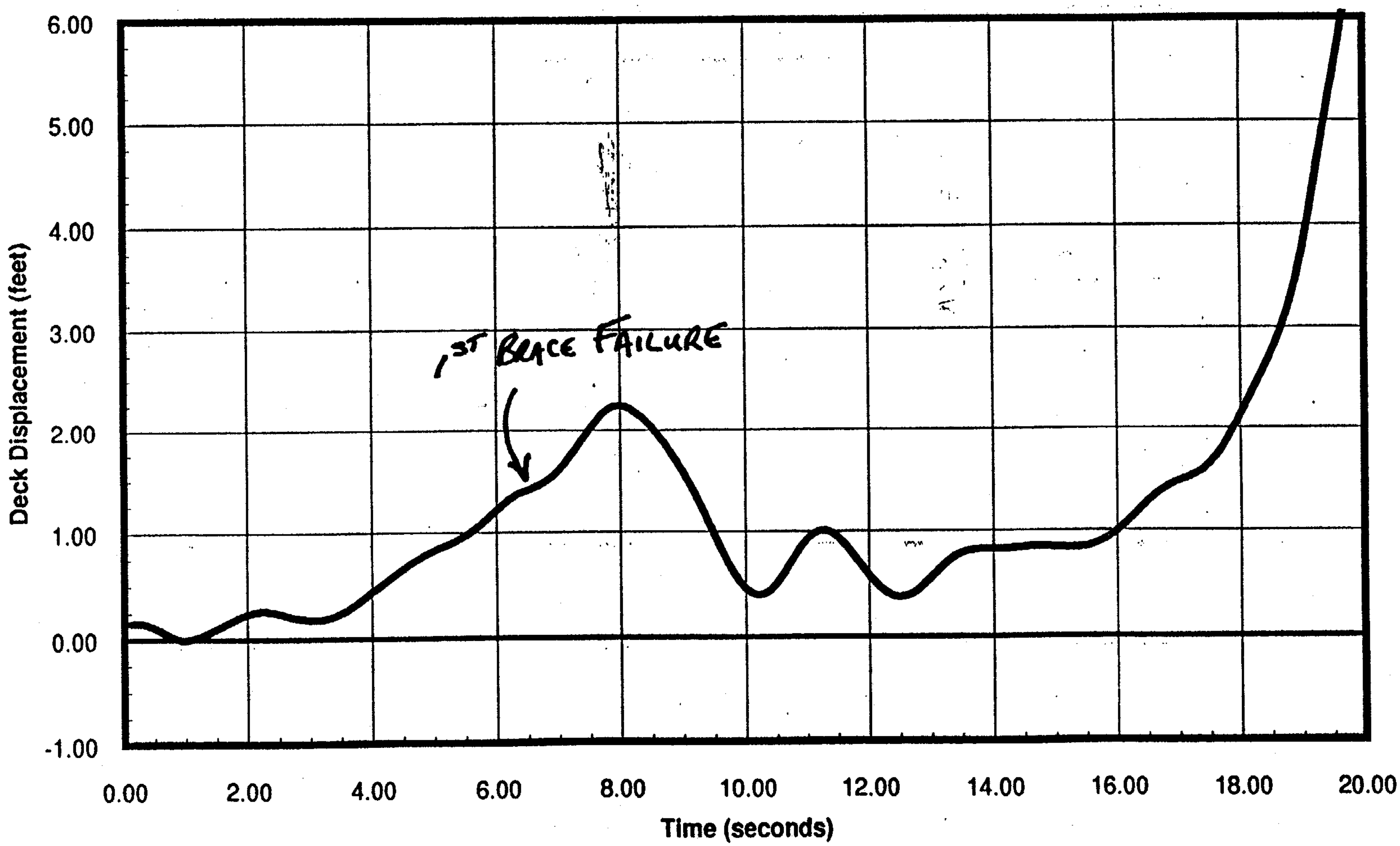


Fig. 6-11

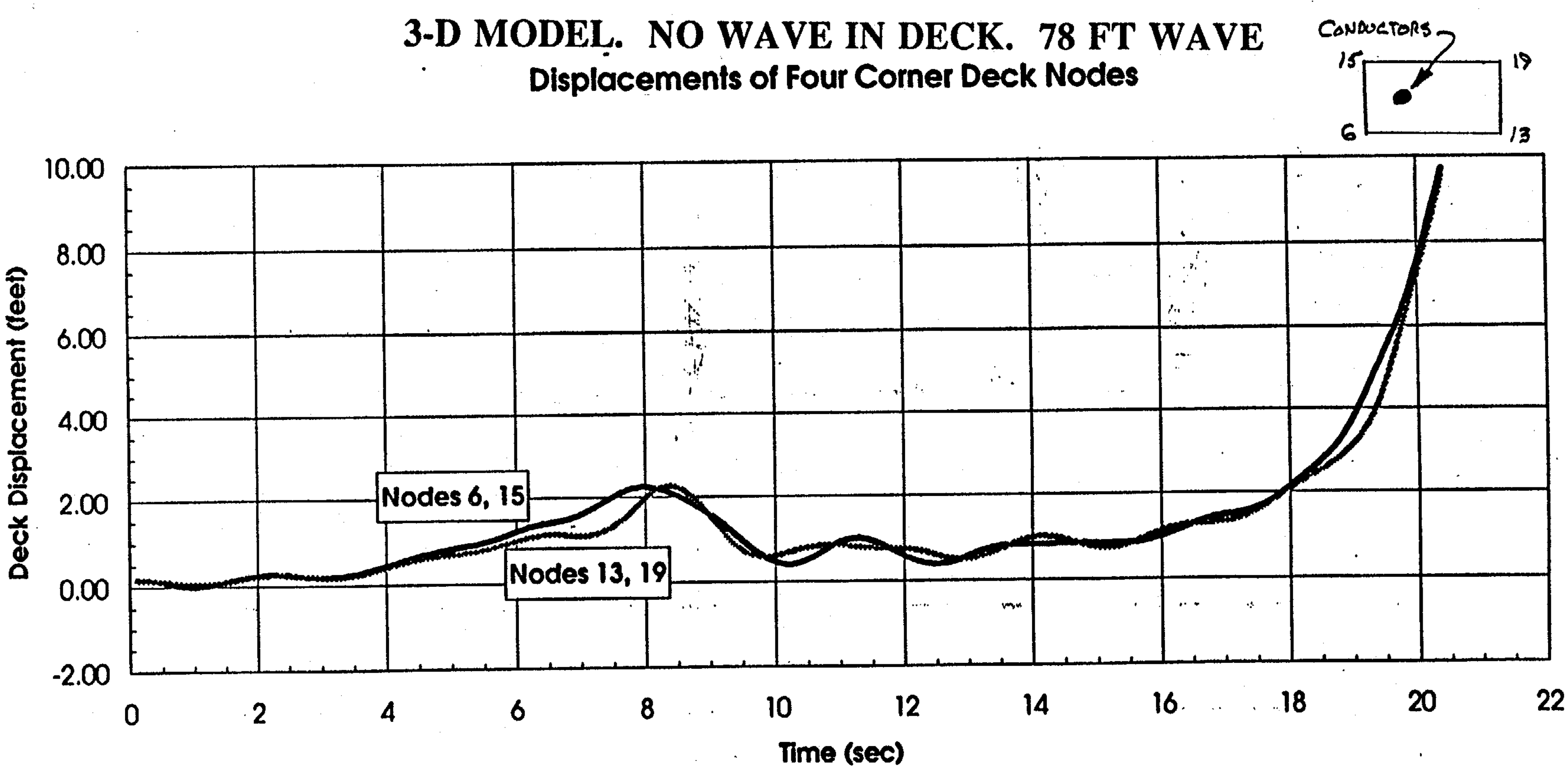

3-D MODEL. NO WAVE IN DECK. 78 FT WAVE
Displacements of Four Corner Deck Nodes
CONDUCTORS
15
19
6
13
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
Deck Displacement (feet)
Nodes 6, 15
Nodes 13, 19
0
2
4
6
8
10
12
14
16
18
20
22
Time (sec)

Fig. 6-12

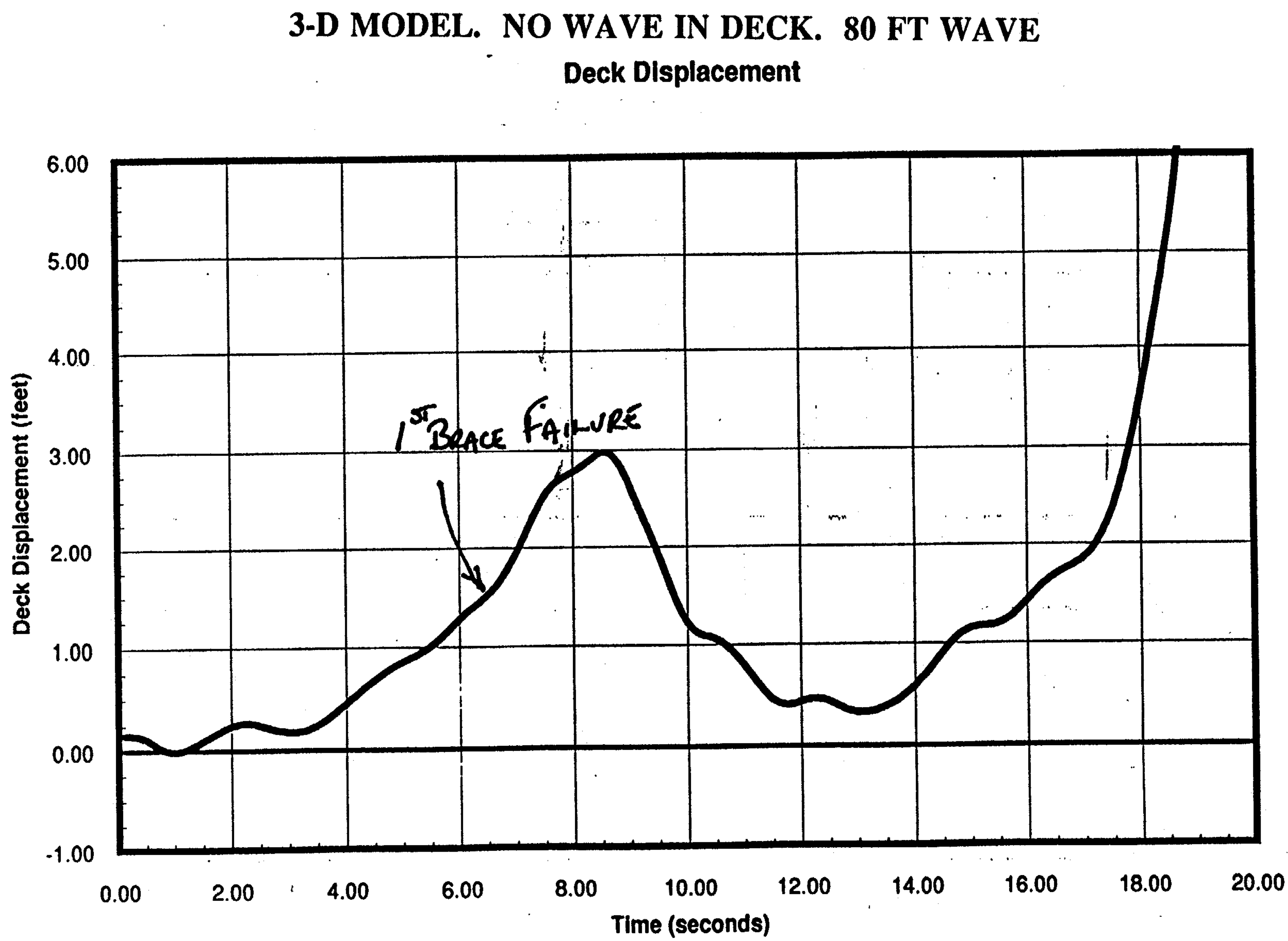
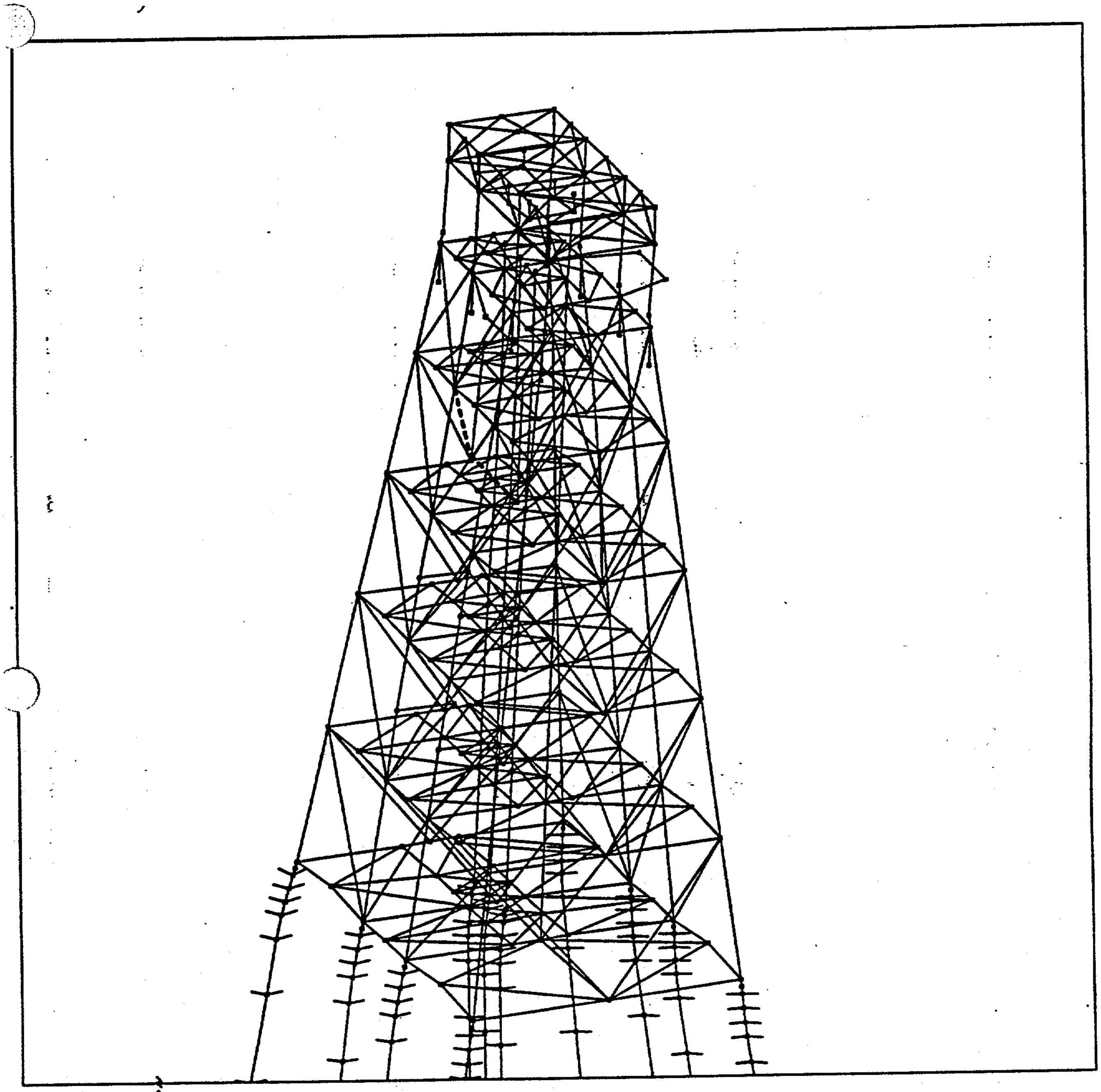

3-D MODEL. NO WAVE IN DECK. 80 FT WAVE
Deck Displacement
6.00
5.00
4.00
3.00
2.00
1.00
0.00
-1.00
Deck Displacement (feet)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
20.00
Time (seconds)
1st BRACE FAILURE

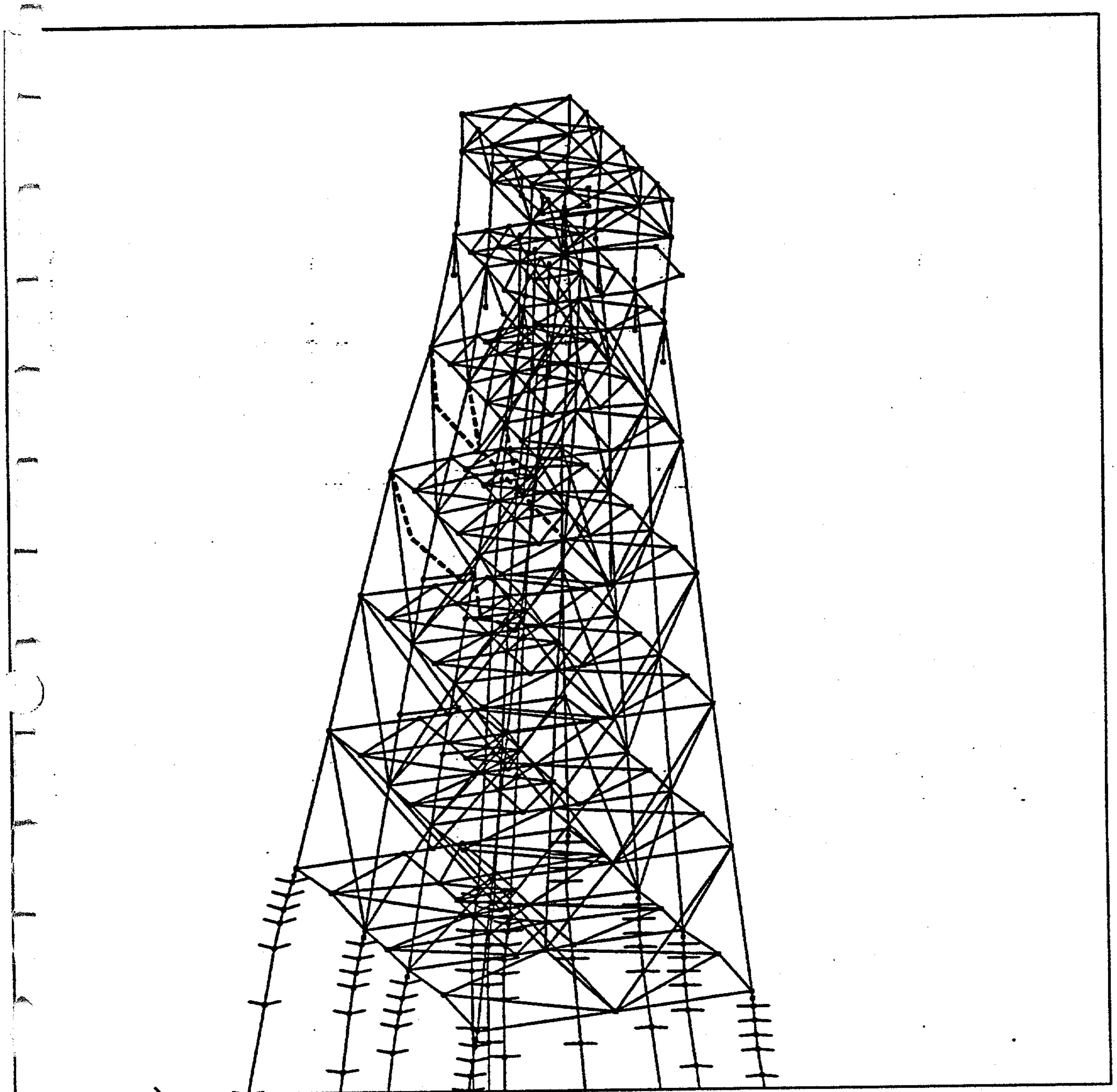
Fig. 6-13



CAP

3-D MODEL. 80 FT WAVE. NO WAVE IN DECK.

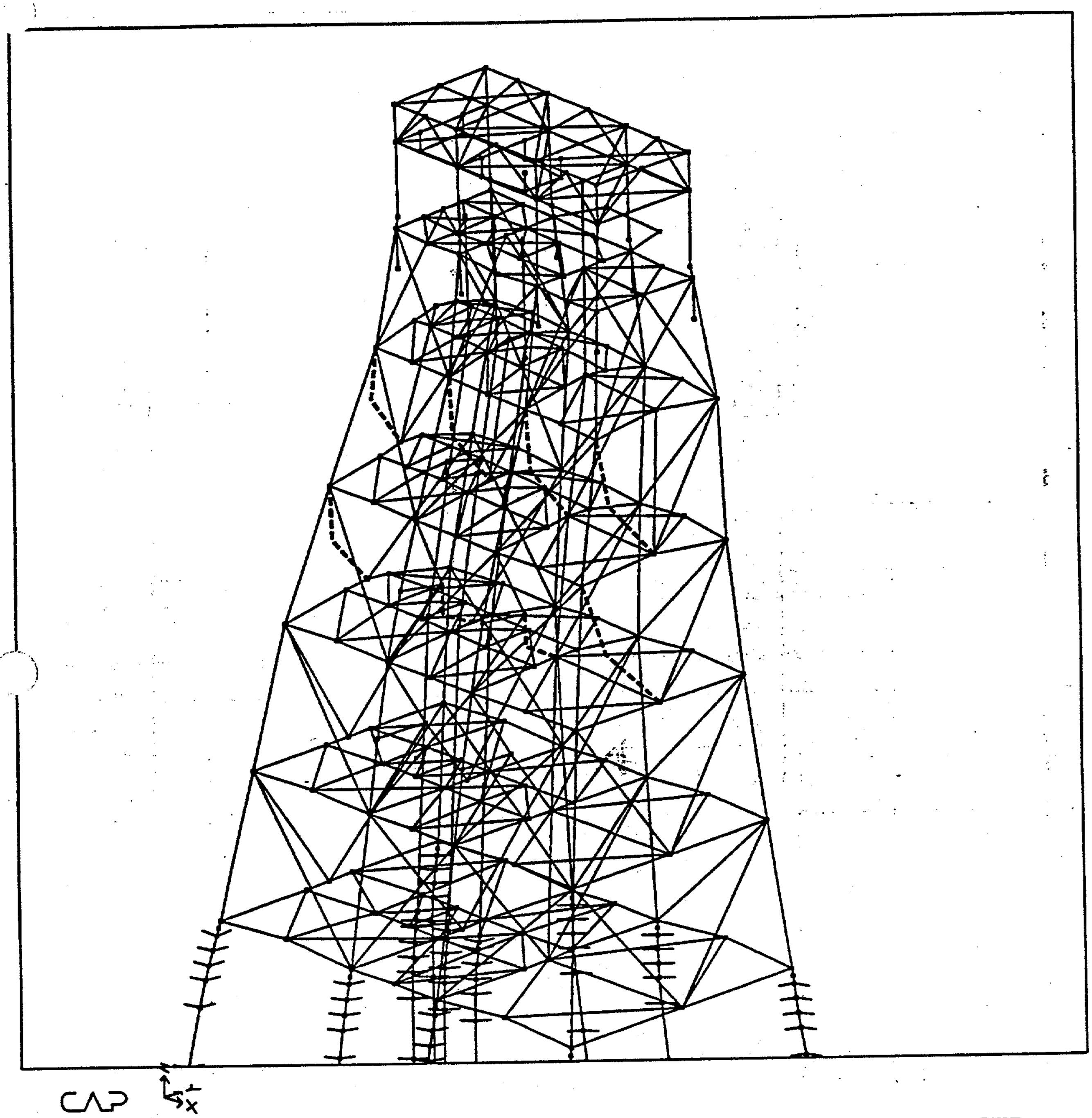
Fig. 6-14



CAP

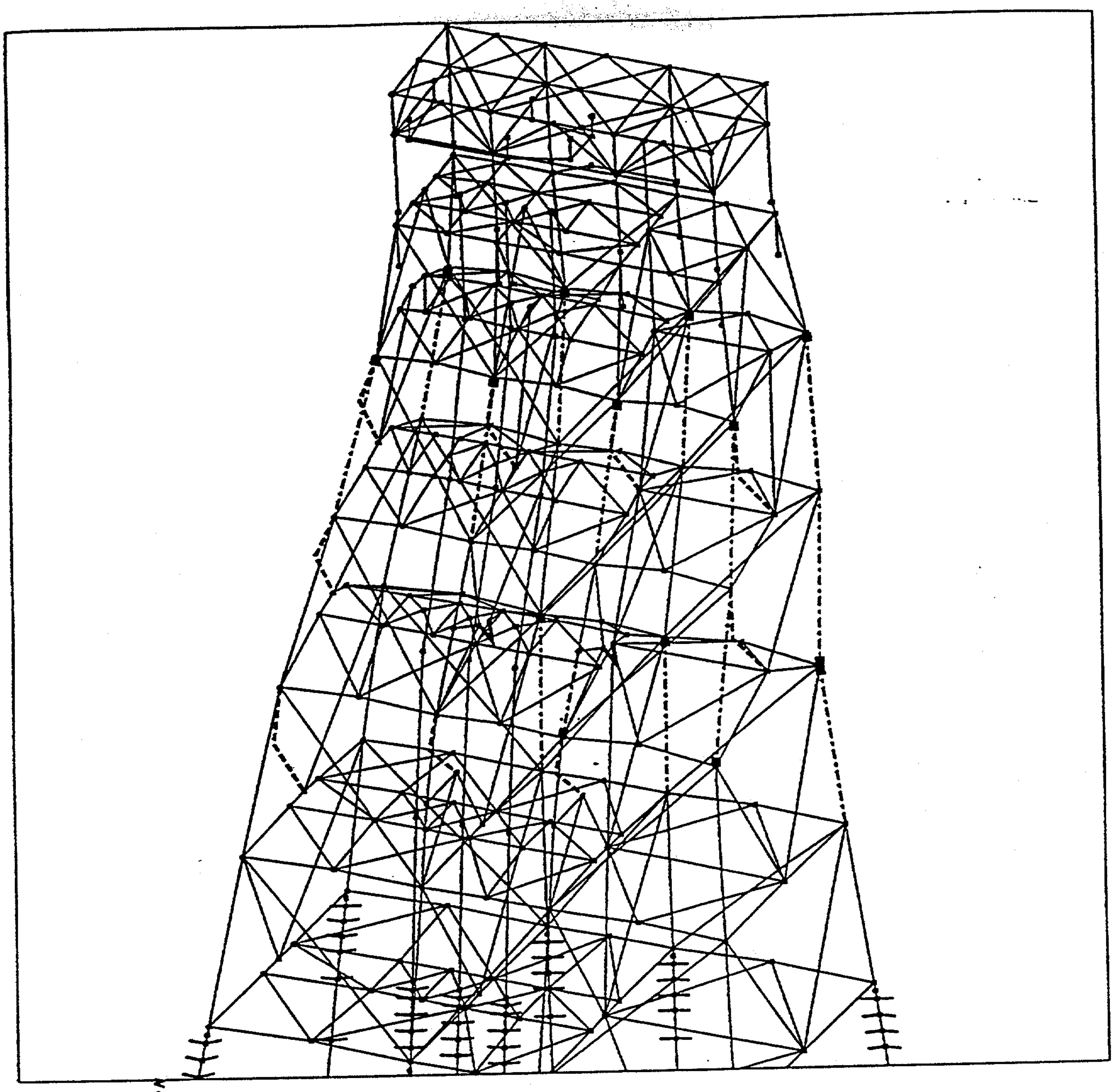
3-D MODEL. 80 FT WAVE. NO WAVE IN DECK.

Fig. 6-15



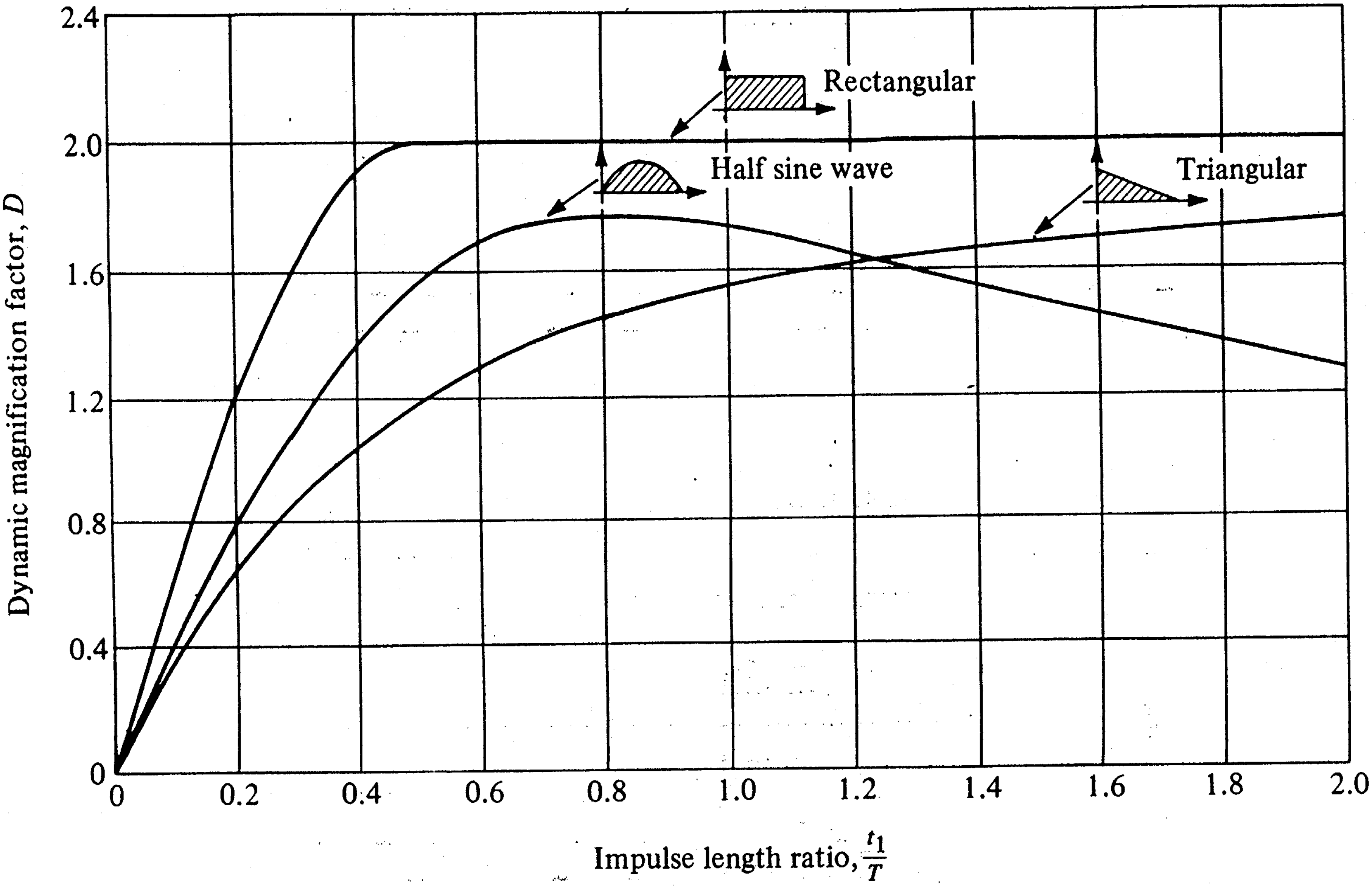
3-D MODEL. 80 FT WAVE. NO WAVE IN DECK.

Fig. 6-16



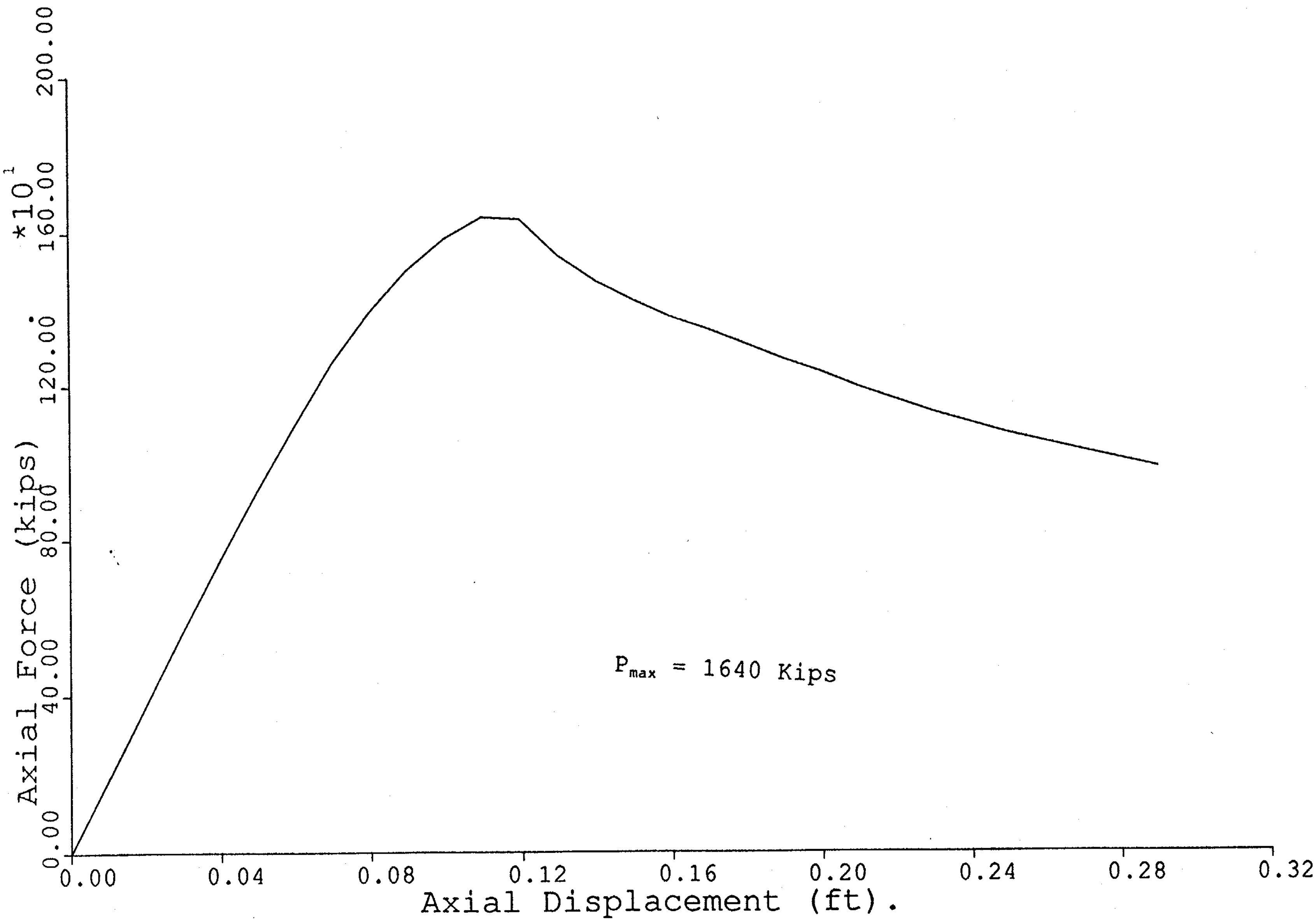
CAP

3-D MODEL. 80 FT WAVE. NO WAVE IN DECK
Fig 6-17



Displacement-response spectra (shock spectra) for three types of impulse.

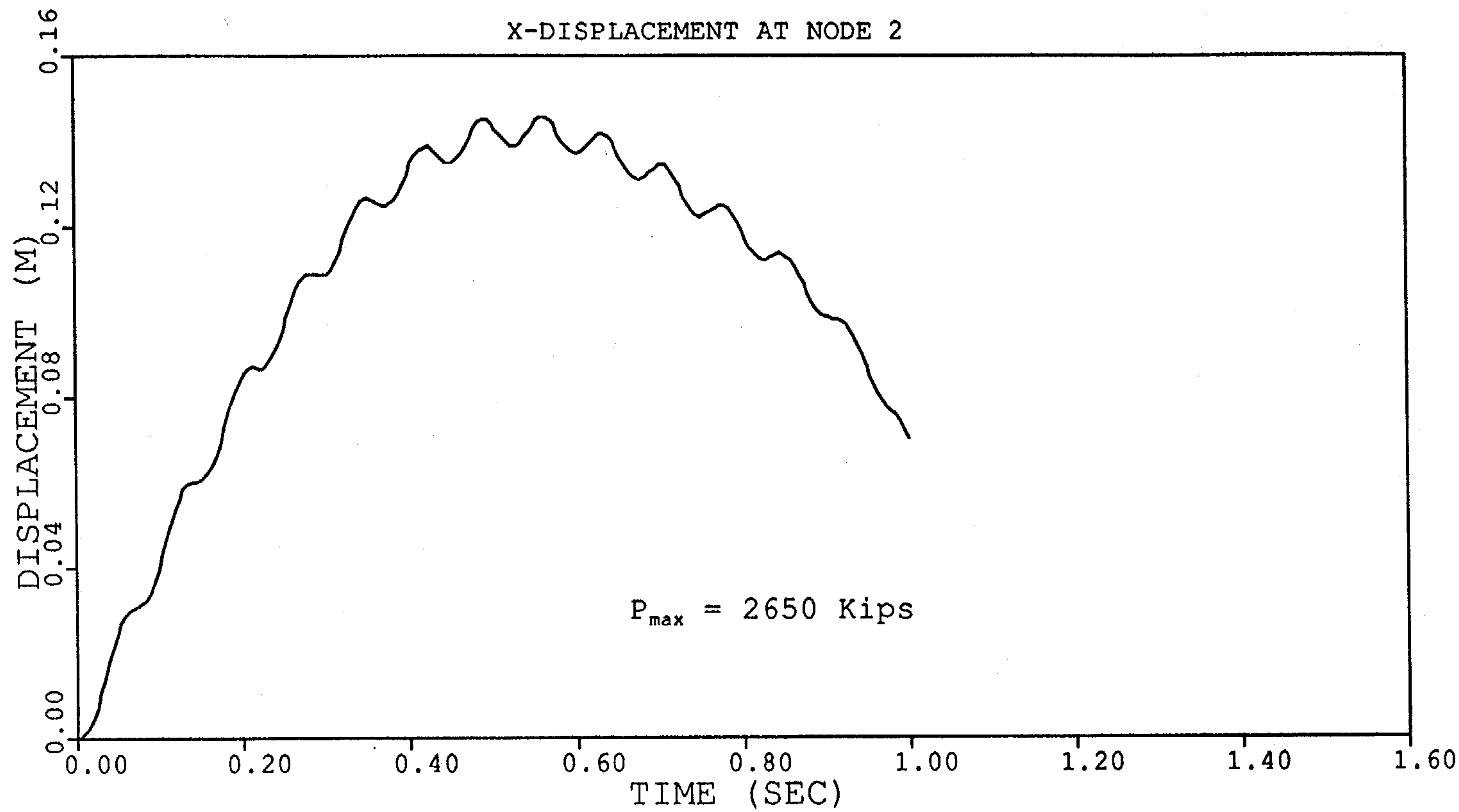
Fig. 6-18



SINGLE BRACE STUDY. STATIC FORCE VS DISPLACEMENT

Fig. 6-19

PMB Engineering Inc., San Francisco - Houston

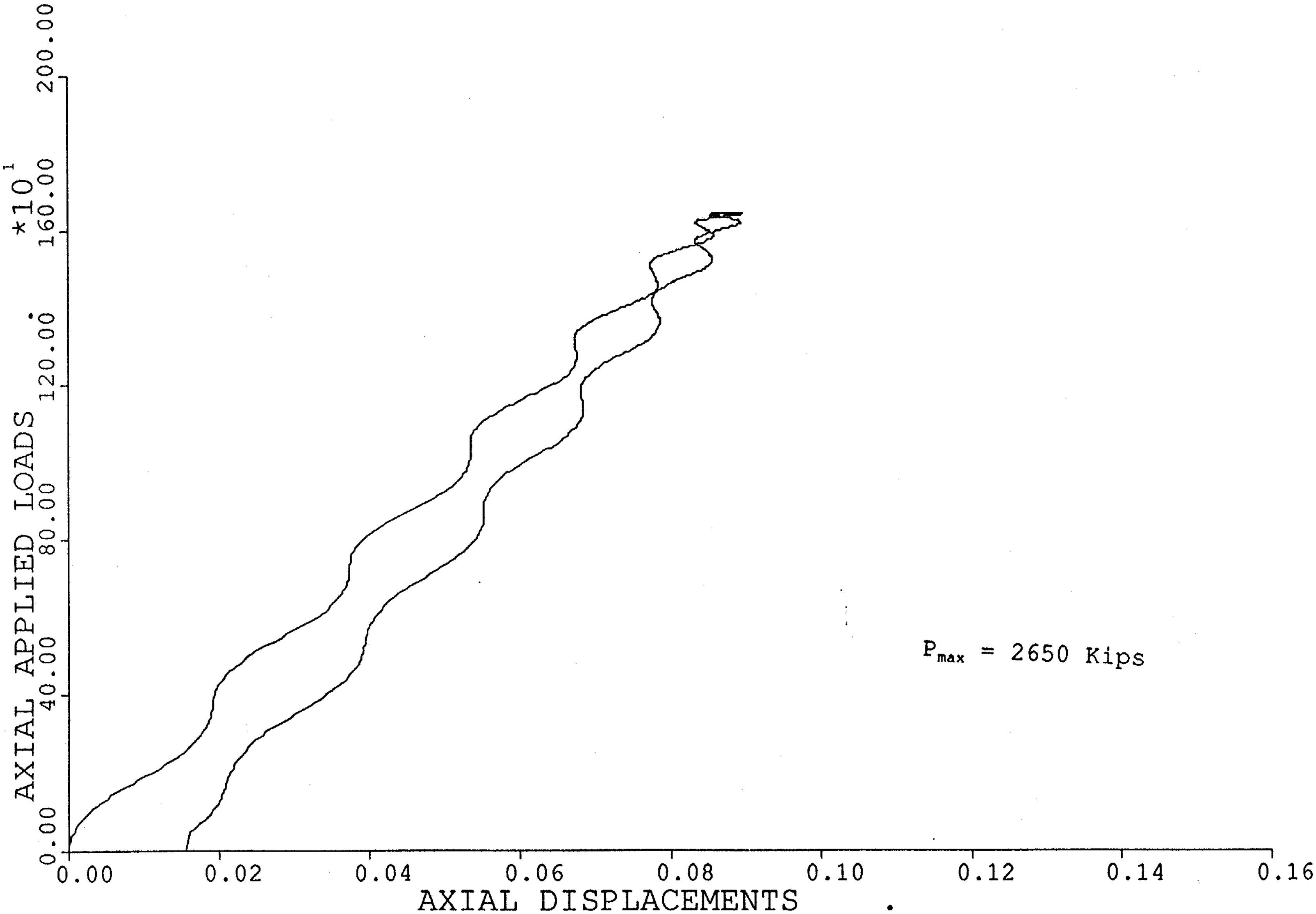


SINGLE BRACE STUDY. DYNAMIC. 2 SEC PERIOD

Axial displacement at End

DATE - 11/17/92 SEAPOST Version 3.02 TIME - 10:55:46

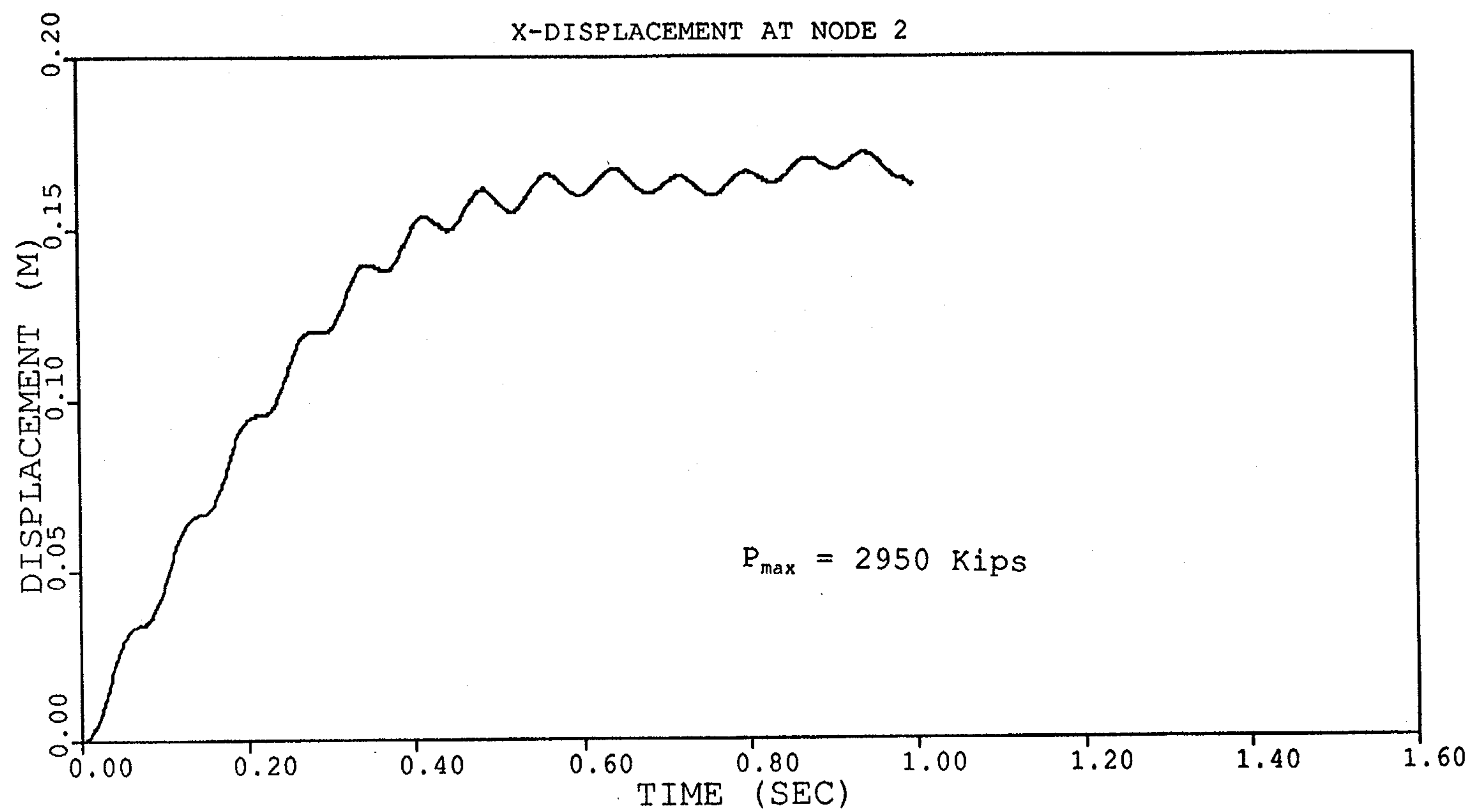
Fig. 6-20



SINGLE BRACE STUDY. DYNAMIC. 2 SEC PERIOD

Fig. 6-21

PMB Engineering Inc., San Francisco - Houston



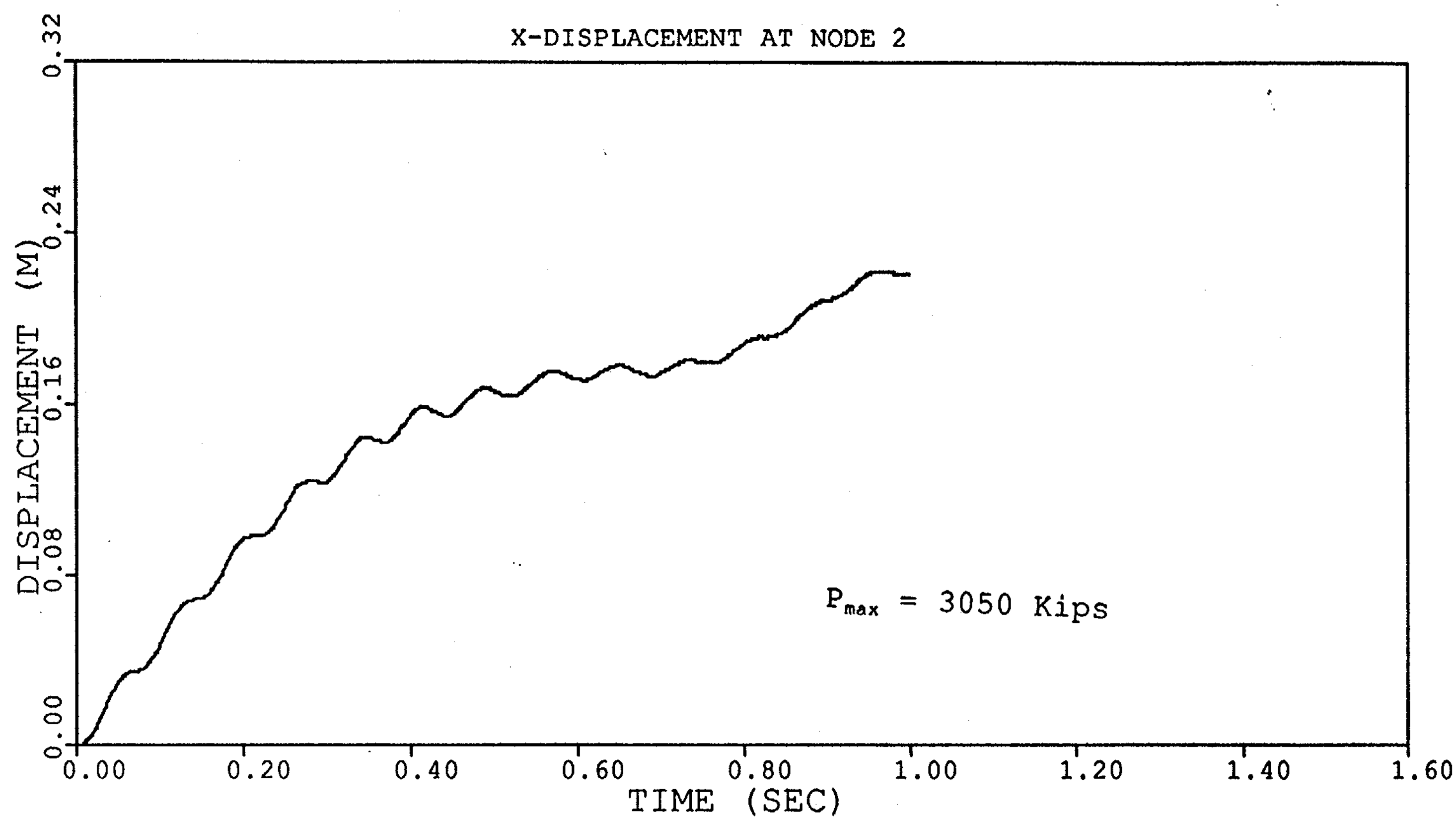
SINGLE BRACE STUDY. DYNAMIC. 2 SEC PERIOD

Axial displacement at End

DATE - 11/17/92 SEAPOST Version 3.02 TIME - 14:32:12

Fig. 6-22

PMB Engineering Inc., San Francisco - Houston



SINGLE BRACE STUDY. DYNAMIC. 2 SEC PERIOD

Axial displacement at End

DATE - 11/17/92   SEAPOST Version 3.02   TIME - 14:39:30

Fig. 6-23

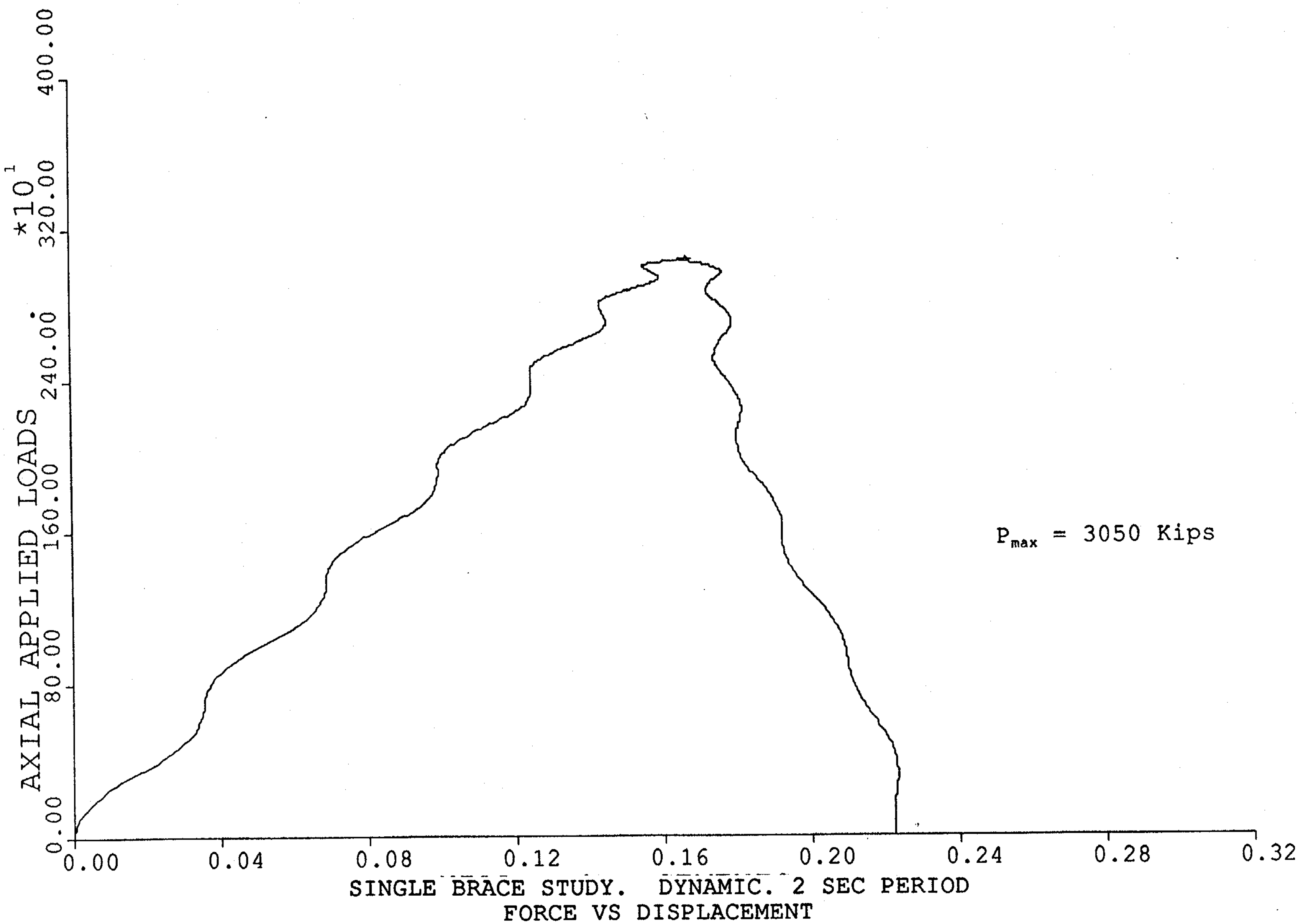

AXIAL APPLIED LOADS
*10¹
0.00
80.00
160.00
240.00
320.00
400.00
P_{max} = 3050 Kips
0.00
0.04
0.08
0.12
0.16
0.20
0.24
0.28
0.32
SINGLE BRACE STUDY. DYNAMIC. 2 SEC PERIOD
FORCE VS DISPLACEMENT

Fig. 6-24

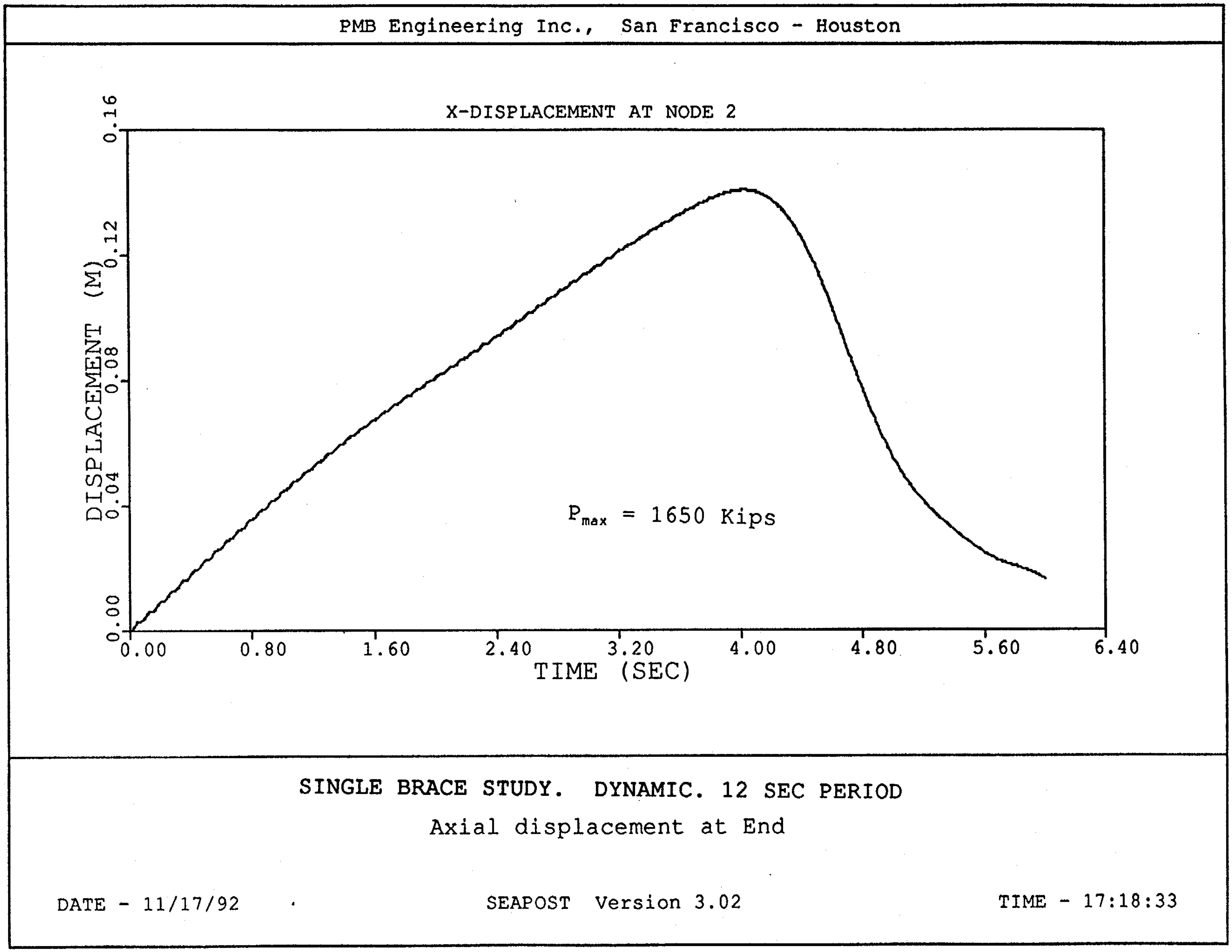

PMB Engineering Inc., San Francisco - Houston
X-DISPLACEMENT AT NODE 2
DISPLACEMENT (M)
0.00
0.04
0.08
0.12
0.16
TIME (SEC)
0.00
0.80
1.60
2.40
3.20
4.00
4.80
5.60
6.40
P_{max} = 1650 Kips
SINGLE BRACE STUDY. DYNAMIC. 12 SEC PERIOD
Axial displacement at End
DATE - 11/17/92
SEAPOST Version 3.02
TIME - 17:18:33

Fig. 6-25

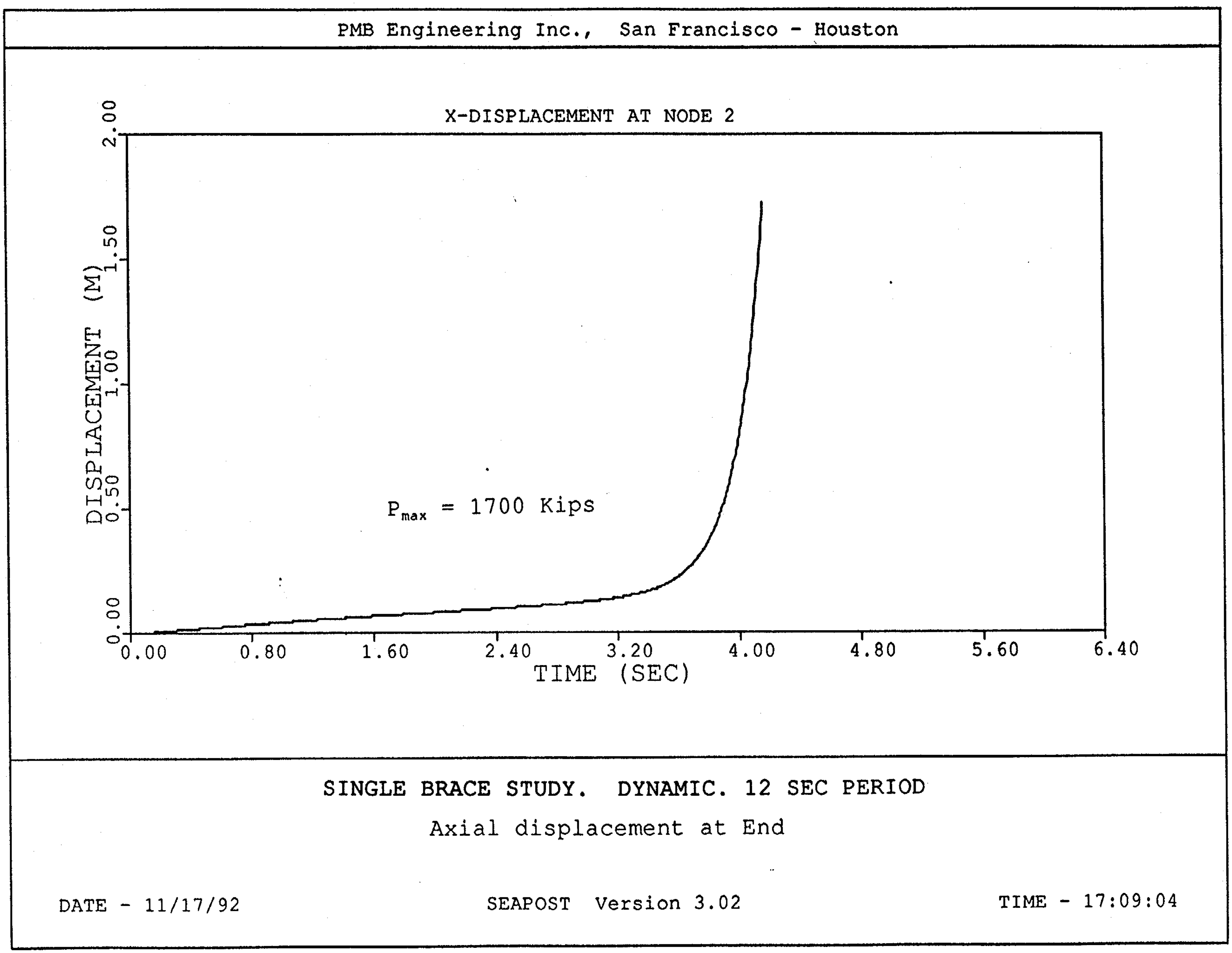

PMB Engineering Inc., San Francisco - Houston
X-DISPLACEMENT AT NODE 2
DISPLACEMENT (M)
0.00
0.50
1.00
1.50
2.00
P max = 1700 Kips
0.00
0.80
1.60
2.40
3.20
4.00
4.80
5.60
6.40
TIME (SEC)
SINGLE BRACE STUDY. DYNAMIC. 12 SEC PERIOD
Axial displacement at End
DATE - 11/17/92
SEAPOST Version 3.02
TIME - 17:09:04

Fig. 6-26

BASE CASE REGULAR WAVES NO DECK LOADS	3 - D	2 - D
1-st Period (sec)	2.14	2.08
STATIC PUSHOVER		
Wave Height	80.0	80.5
Fstat (Total)	4900	1390
Failure Mode *	J	J
DYNAMIC WAVES		
1) 1-st Event :		
Wave Height	Not Available	78.0
Fdyn (Total)		1330
Failure Mode *		J
2) Collapse :		
Wave Height	78.0	79.0
Fdyn (Total)	4700	1350
Fdyn/Fstat	0.96	0.97
Failure Mode *	J	J

Table 6 - 1. Comparison of 2D, 3D Model Strengths

* J = Jacket
P = Portal

BASE CASE REGULAR WAVES WITH DECK LOADS		3 - D	2 - D
Deck Wave Load		SHELL	API
1-st Period (sec)		2.14	2.08
STATIC PUSHOVER Wave Height Fstat (Total) Failure Mode *		 73.0 4900 	 73.5 1400 P, J
DYNAMIC WAVES 1) 1-st Event : Wave Height Fdyn (Total) Failure Mode * 2) Collapse : Wave Height Fdyn (Total) Fdyn/Fstat Failure Mode *		 Not Available NOT RUN 	 72.0 1280 P 75.0 1430 1.02 J, P

Table 6 - 2. Comparison of Strength Using SHELL and API Deck Loads

* J = Jacket
P = Portal

Table 6-3 Results of Brace Dynamic Capacity Study

Period of Load (sec)	Collapse Load (kips)	Applied Load Period/ First Mode Period
Static	1640	Inf.
12.0	1675	8.8
2.0	3000	1.5

Section 7
Irregular Multiple Wave Dynamic Capacity

7.1 APPROACH

Section 6 described the studies that were carried out on the 2-D and 3-D models to determine the collapse of the jacket when a dynamic analysis was performed using two cycles of regular waves. These studies were extended by simulating the response of the 2-D jacket to more complex wave loadings. The object of these studies, like those of the regular waves, was to determine whether a static pushover analysis gives the same results as those determined by a full dynamic analysis.

In these studies wave loads were calculated without waves in the deck, since if dynamic effects were present, they would likely appear for the impulse-type loading of waves in the deck.

The first study used a wave history in which the waves increased and then decreased again in a formalized way. It commenced with three 70 ft waves that are below the height to cause any member failures. The waves then increased linearly in height to a maximum crest and then decreased again to 70 ft, ending with three cycles of 70 ft waves again. Two simulations were made, for the 2-D model, one with the maximum wave of 80 ft (called the 70-80-70 sequence, the other with 81 ft (70-81-70). One was made with the 3-D model with the maximum of 81 ft (70-80-70). The periods of all waves were 13.0 sec, and currents were included as for the regular waves. A resulting surface elevation history is shown in Figure 7-1.

The second study consisted of a set of measured surface elevations from Hurricane Camille. Four groups of waves were selected in pairs, waves being chosen to have rather long and rather short crests. These waves are shown in Figures 7-2 through 7-5. Since none of the waves in Camille was high enough to cause member failures, the measured elevations were scaled up with a constant factor for each wave group until platform failure occurred.

7.2 SYNTHETIC IRREGULAR SEA CAPACITY

In the regular wave studies described in Section 5, when the lowest wave that resulted in collapse was examined, it was found that collapse never occurred on the first wave cycle. Significant damage occurred in this cycle, but, due to the short peak loading time relative to the first natural period of the softened structures after a failure, the structure did not have time to fully react to the applied load, and the structure survived into the next wave, when collapse occurred. (If a sufficiently large wave had been applied, collapse could have been initiated in the first wave crest, but this would have been a larger wave than the lowest wave that causes collapse.) The purpose of this study was to see whether the reduction in load in the wave crest following the largest crest, could allow the jacket to survive the wave sequence.

The waves were chosen to approximate the set of the 5 highest waves in a storm [Stewart, 1993]. Stewart describes various statistical processes to rank the five highest waves. The ratio of the wave heights of these five waves to the largest, as calculated by Stewart, is shown in the second column of Table 7-1. These values depend on the length of storm and dominant periods, so are independent of site. The third and fourth columns show the wave heights and their proportion to the largest, for the 70-80-70 group used in this current study. It can be seen that these waves approximate the expected five largest waves in a storm quite well, although the waves in a real storm would not necessarily occur in sequence as assumed here.

By definition, in a real storm the largest wave is always followed by smaller waves. The second-highest wave may have occurred previously, or it may follow the maximum wave. Generally it will not occur as the crest immediately following the highest (see Section 7.4). If the structure can survive the largest wave without collapse, it may survive the storm provided the weakened structure can survive the next highest wave, and the subsequent waves ranked statistically downward. It was decided to use as the largest wave the wave that caused member damage, but did not result in collapse at the first crest, and see whether the jacket could ride out the following waves that were decreasing in height in a manner approximating the heights of the largest waves in the storm.

Two wave sequences were studied. The first, 70-80-70, had three crest of 70 ft, three increasing to 80 ft, three decreasing back to 70 ft, and then a total of three at 70 ft. (Figure 7-1). The 70-81-70 sequence was identical, except for the height of the largest wave. These waves were created from stream function wave profiles with height equal that of the largest wave. The free surface profile was developed for the full eleven waves, and the points on this profile were then scaled down by the appropriate factor.

A Fourier transform of the resulting surface history gave a set of wave components with periods ranging from 143 sec. to 2.6 sec. Table 7-2 shows these components for the 70-80-70 wave set. The water particle motions from these wave components were then superimposed (and combined with current), using Wheeler stretching.

It was, however, not possible to follow this recipe exactly, because there would have been quite significant differences in the applied wave loads from the waves in the irregular wave sequence and waves of the same height computed from stream function theory. If the surface elevation history from a stream function wave is Fourier-transformed into independent wave components and these are combined with Wheeler stretching to give water particle motions, the resulting wave loads will be rather different from those of the original stream function results. Figure 7-6 shows the horizontal water particle velocity profile for a 70 ft wave from stream function theory and from irregular wave theory (combining components). It is seen that there is considerable difference in the water particle velocities in regions that are important to total jacket wave loading, irregular wave theory giving a low value relative to stream function.

In order to lessen the effect of this inconsistency, the water particle motions were increased in the irregular wave simulations by increasing the wave kinematics factor from 0.75 used in stream function runs to 0.89 for irregular wave runs. This factor was determined by comparing the wave load on the jacket from a true stream function wave, and from the wave component approach. The surface history from a 75 ft wave was derived from stream function theory. This was Fourier-transformed into wave components and these were combined using Wheeler stretching. The resulting wave forces at the crest were compared with those from stream function theory. It was found that by multiplying the wave kinematics in the irregular wave theory by 0.89 it gave the same total wave plus current load as the stream function wave plus current case when a multiplier of 0.75 was used. All irregular waves in this project were therefore simulated using a wave kinematics factor of 0.89, while stream function waves used 0.75. This procedure is shown schematically in Figure 7-7.

Figure 7-8 shows the deck displacement history from the 70-80-70 waves. The first nonlinear event occurred, a failure of brace B3, at time 71.3 when the 80 ft crest (the sixth) passed by the jacket. There were no other events at this crest, but at the following crest, even though it is only 76.7 ft high, the weakened jacket experienced further brace failures followed by leg failures and finally collapse.

Increasing the maximum wave height to 81 ft, (70-81-70) resulted in about 15% more compressive deflection in brace B3 in the first crest, but the failure was generally quite similar to that for the smaller wave sequence. (Figure 7-9.)

When this is compared with the previous regular waves, no great difference is observed. In the regular wave sequence a wave of 80 ft caused collapse in the second crest, after member failures had occurred in the first. The reduction in height of the crest following that at which member failures first occurred, did not provide enough relief for the jacket to get through the second crest without collapse.

Results from this series of studies are summarized in Table 7-3. A description of the quantities shown in the table is given in Section 6.3 (for Table 6.1.)

It can be stated therefore that the static pushover analysis appears to provide a quite accurate measure of the capacity of the structure subjected to dynamic waves, ranked with heights approximating the highest five wave in the storm.

7.3 MEASURED IRREGULAR SEA CAPACITY

Four wave sets were selected from Hurricane Camille data. Each wave set included three wave crests, and commenced at a trough, as was done with the regular and synthetic waves, to keep transients small. Waves sets N1 and N2 had narrow steep crests for the middle, highest wave, while W1 and W2 were rather broader. These waves are shown in Figures 7-2 through 7-5.

Similar to the synthetic regular seas, these studies were also made without deck wave loads.

The water particle kinematics were computed from irregular wave theory, using wave components determined from the surface histories by Fourier decomposition. Wheeler stretching was used, and the kinematics factor was increased to 0.89 (for irregular waves) as discussed in Section 7.2. In separate runs, the heights of the wave components were scaled up progressively, resulting in higher wave crests for all waves in each wave set. Depending on the wave set, the multipliers ranged from about 1.2 to 2.2. The actual simulation is identified by the crest height of the middle (largest) wave in the set. Thus N1-69.0 is the wave set created from measured waves N1, scaled up to give a middle wave with crest elevation 69.0 ft. The wave height that resulted in a failure of one brace was identified for each of the four measured wave groups, and the wave height resulting in collapse.

Scaling the wave component amplitudes by a constant creates a free surface history that is scaled up from the original, increasing the heights of each wave the same constant. Although this procedure preserves the randomness of real seas, it cannot be expected to simulate accurately the increasing skewness of the free surface elevation as the scaling factor is increased. (In real waves, the crests are higher than the depths of the troughs, this effect increasing with wave height.) Thus, creating scaled-up waves by this process might not give quite the same results as choosing measured waves of the same height. This latter alternative did not exist, since waves recorded during this storm were not large enough to fail the structure.

Figure 7-10 through 7-12 for wave set N1 shows a typical set of deck displacements as the scaling factor increases. At first sight there appears to be considerable transient present in these responses. However, it has not died out significantly after 35 sec, or 17 cycles of the natural period of 2.1 sec. With 5% damping, a transient would have reduced to less than 1% of its starting value, so this higher frequency response is response to higher frequencies in the complex wave load history.

N1-62.3 had no member failures. N1-68.0 had 1 brace failure (B3) in the middle crest, but no jacket collapse occurred. N1-69.0 had braces B3 and B5 fail in the middle crest, and in the last crest failure of the jacket legs resulted in collapse of the structure as indicated in Figure 7-12.

Table 7-4 summarizes the results of this series of tests. A description of the quantities shown in the table is given in Section 6.3 (for Table 6-1). The waves that cause first brace failures have total applied loads a little smaller than for the static pushover for the base case (stream function wave theory.) The wave loads that cause collapse have total loads that are within 4% the same as those for the base case, after the dynamic amplification has been factored into them.

The conclusion from this series of tests is that a measured sea, scaled up to cause collapse, is not significantly different from regular waves. Further, for this structure, collapse is adequately predicted for such seas by static pushover analysis.

7.4 COLLAPSE AND WAVE HEIGHT SEQUENCE

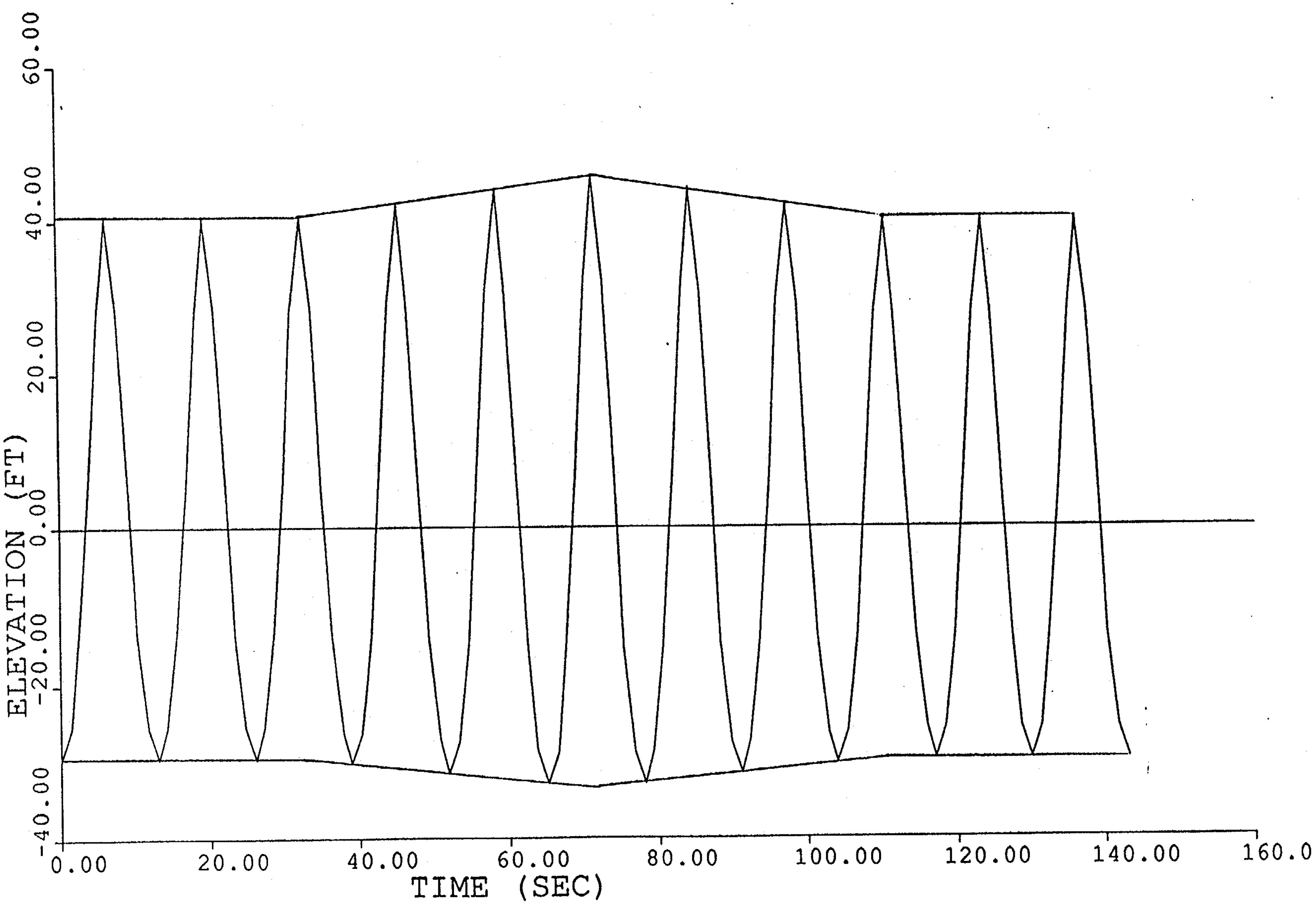
It is sometimes hard to define what the collapse wave height is in a multi-wave environment, since collapse depends on the sequence of wave heights.

If the wave that first causes damage is the largest in the storm, or the largest that the platform is expected to experience in its lifetime, the following waves will be smaller, and could be below the size needed to cause collapse.

Most of the test cases studied were regular waves and thus had two wave crests of equal height (Section 6). In those simulations, the jacket failed on the second crest, after surviving the first. In the irregular waves studied in Section 7, the wave crests vary in height.
In the synthetic irregular waves, the second largest crest was about 95% of the largest, (Figure 7-1) and the results were not greatly different from the regular wave results, the wave height of the largest wave being similar to the static collapse wave height.

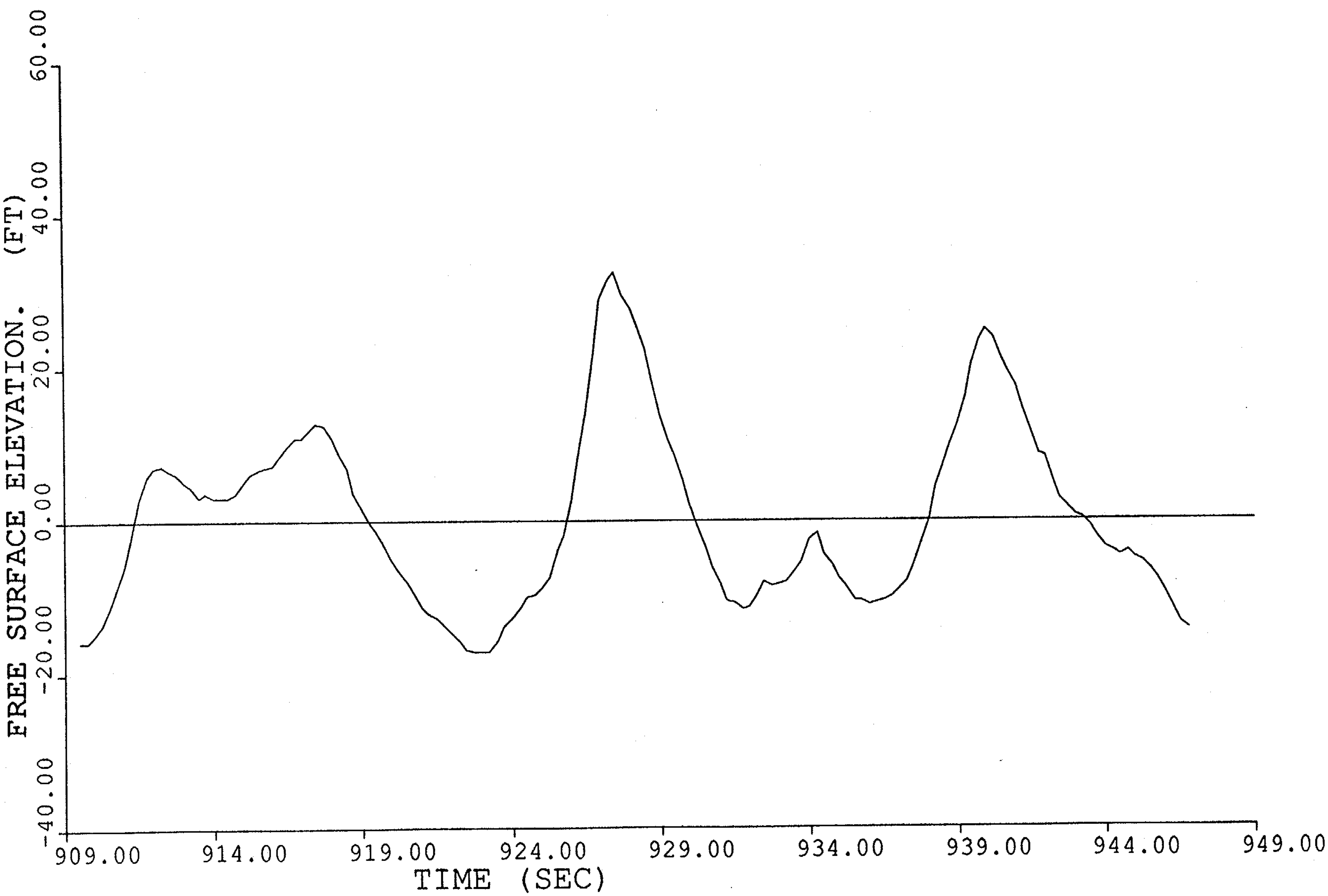
The measured irregular waves all had the second largest crests substantially less than the first (78%, 28%, 67%, 72%, for waves N1, N2, W1, W2, respectively) and the structure again collapsed during these follow-up smaller waves. In all these cases, the height of the largest wave was almost exactly the height that caused collapse statically.

This suggests that a fairly critical definition can be made of the height of the collapse wave in multi-wave conditions, without having to consider the heights of other smaller waves that may follow the largest wave. Further, for the base case configuration of the jacket, this wave height is virtually the same as that for collapse under static conditions.



FREE SURFACE ELEVATION. 70FT-80FT-70FT

Fig 7-1



MEASURED IRREGULAR WAVE N1

Fig. 7-2

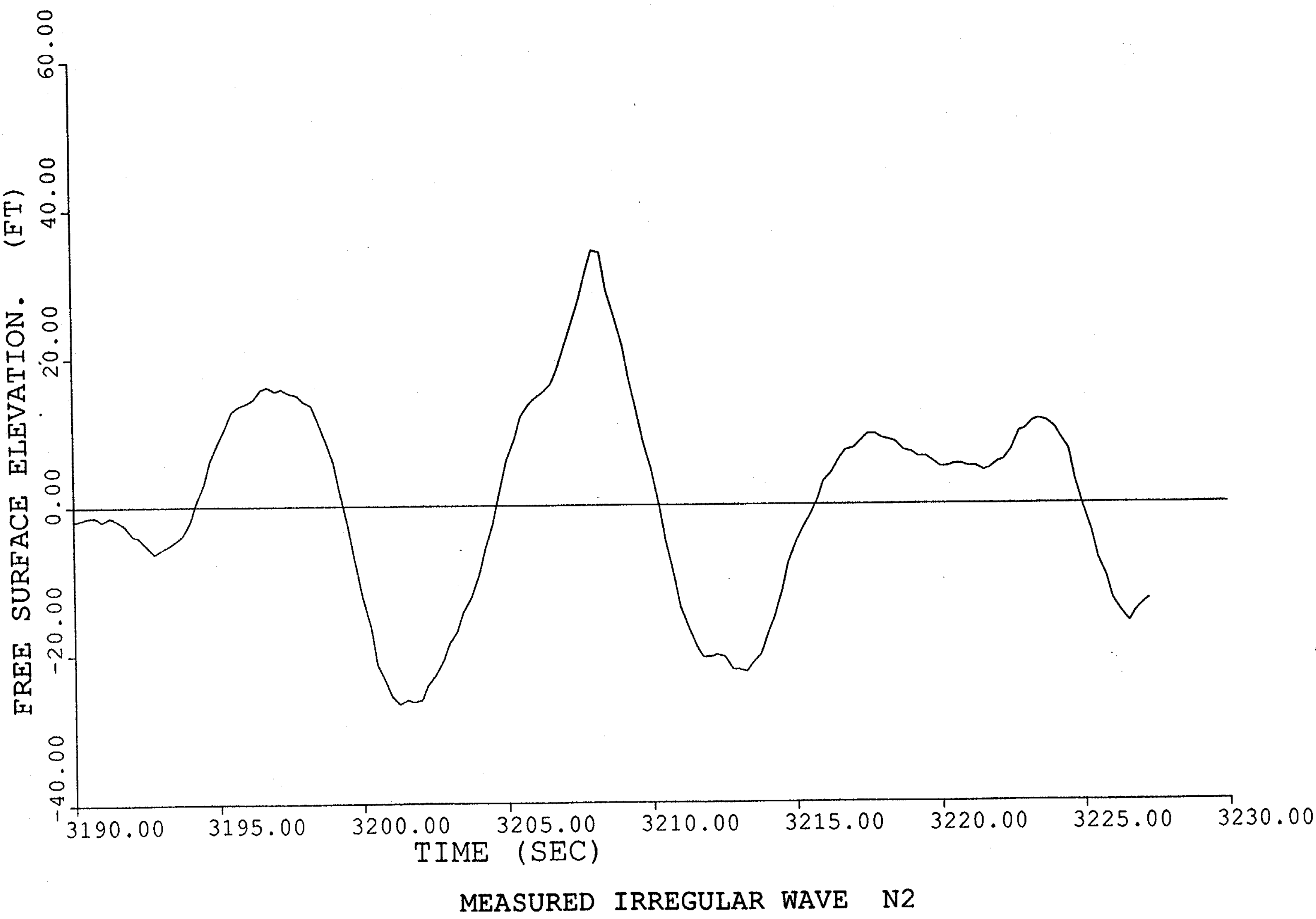
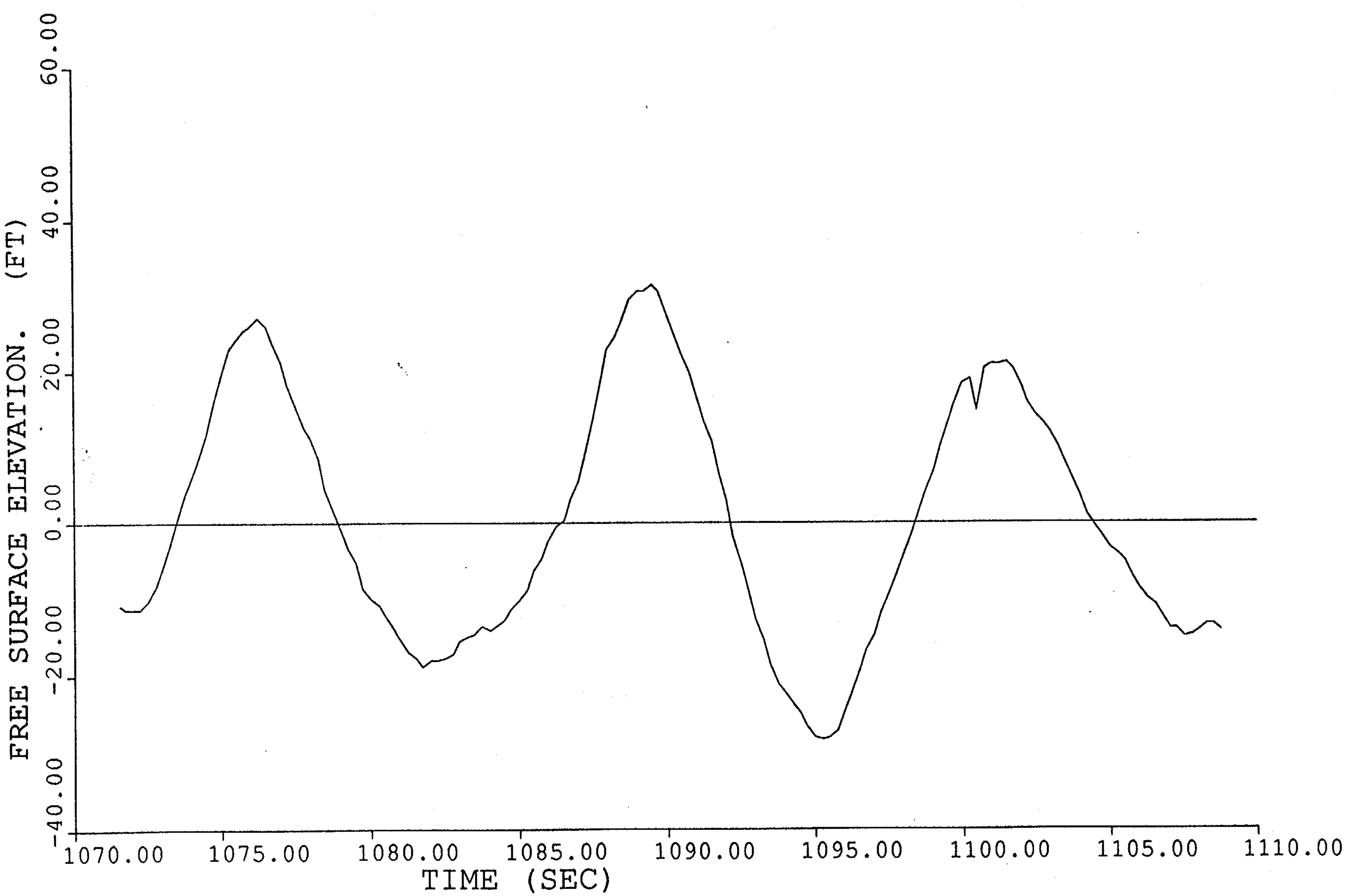
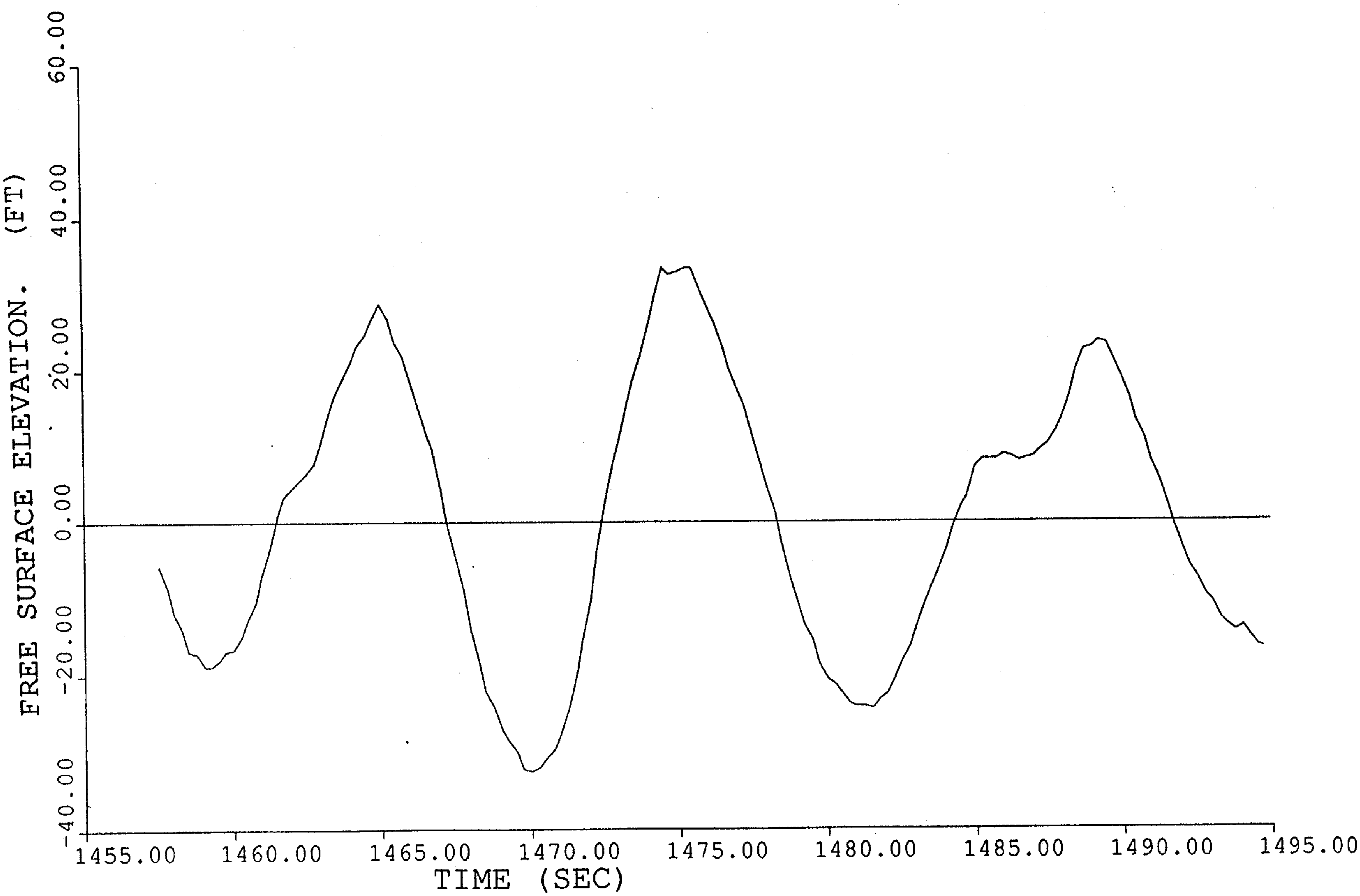

60.00
40.00
20.00
0.00
-20.00
-40.00
FREE SURFACE ELEVATION. (FT)
3190.00
3195.00
3200.00
3205.00
3210.00
3215.00
3220.00
3225.00
3230.00
TIME (SEC)
MEASURED IRREGULAR WAVE N2

Fig. 7-3



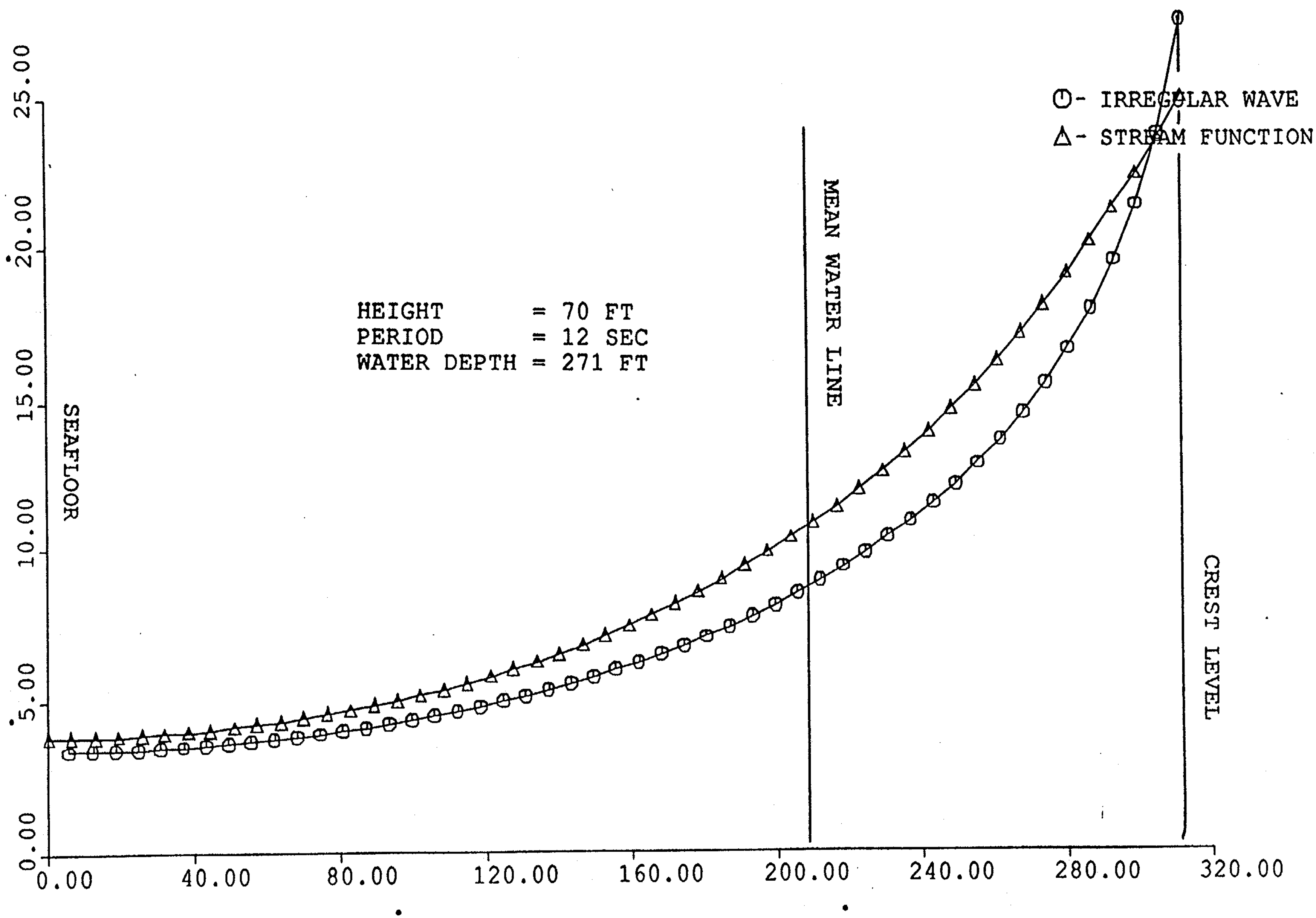
MEASURED IRREGULAR WAVE W1

Fig. 7-4



MEASURED IRREGULAR WAVE W2

Fig. 7-5



STREAM FUNCTION AND IRREGULAR WAVE KINEMATICS
HORIZONTAL VELOCITY UNDER CREST

Fig. 7-6

STREAM FUNCTION WAVE THEORY

↓

WAVE KINEMATICS

↓

KINEMATICS FACTOR = 0.75

↓

WAVE LOADS

||

EQUAL TOTAL WAVE LOADS

||

WAVE LOADS

↑

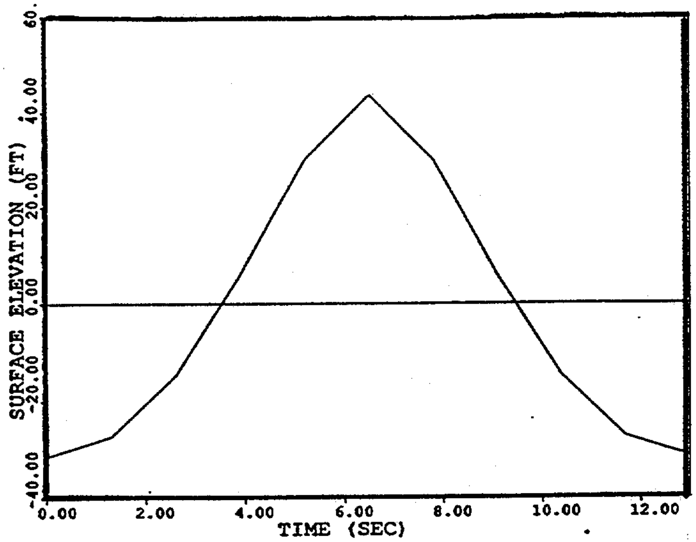
KINEMATICS FACTOR = 0.89

↑

WAVE KINEMATICS

↑

IRREGULAR WAVE THEORY (WHEELER STRETCHING)



FREE SURFACE ELEVATION

↓

FOURIER TRANSFORM

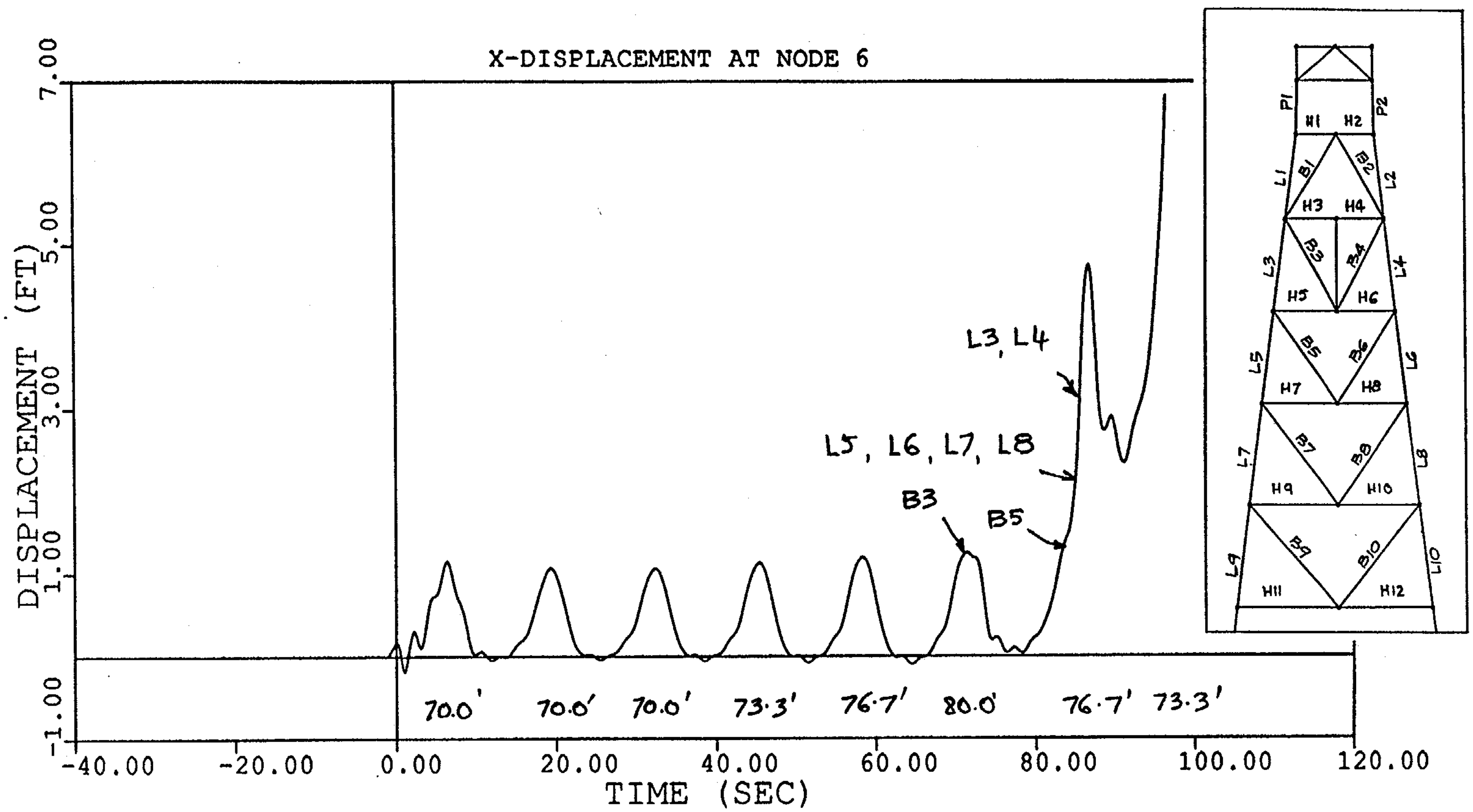
↓

Period (sec)	Height (ft)
13.0	72.10
6.50	11.64
4.33	2.67
3.25	0.80
2.60	0.22

Wave Components Derived from Stream Function 75ft Wave

Fig. 7-7 Kinematics Factor for Irregular Waves

PMB Engineering Inc., San Francisco - Houston



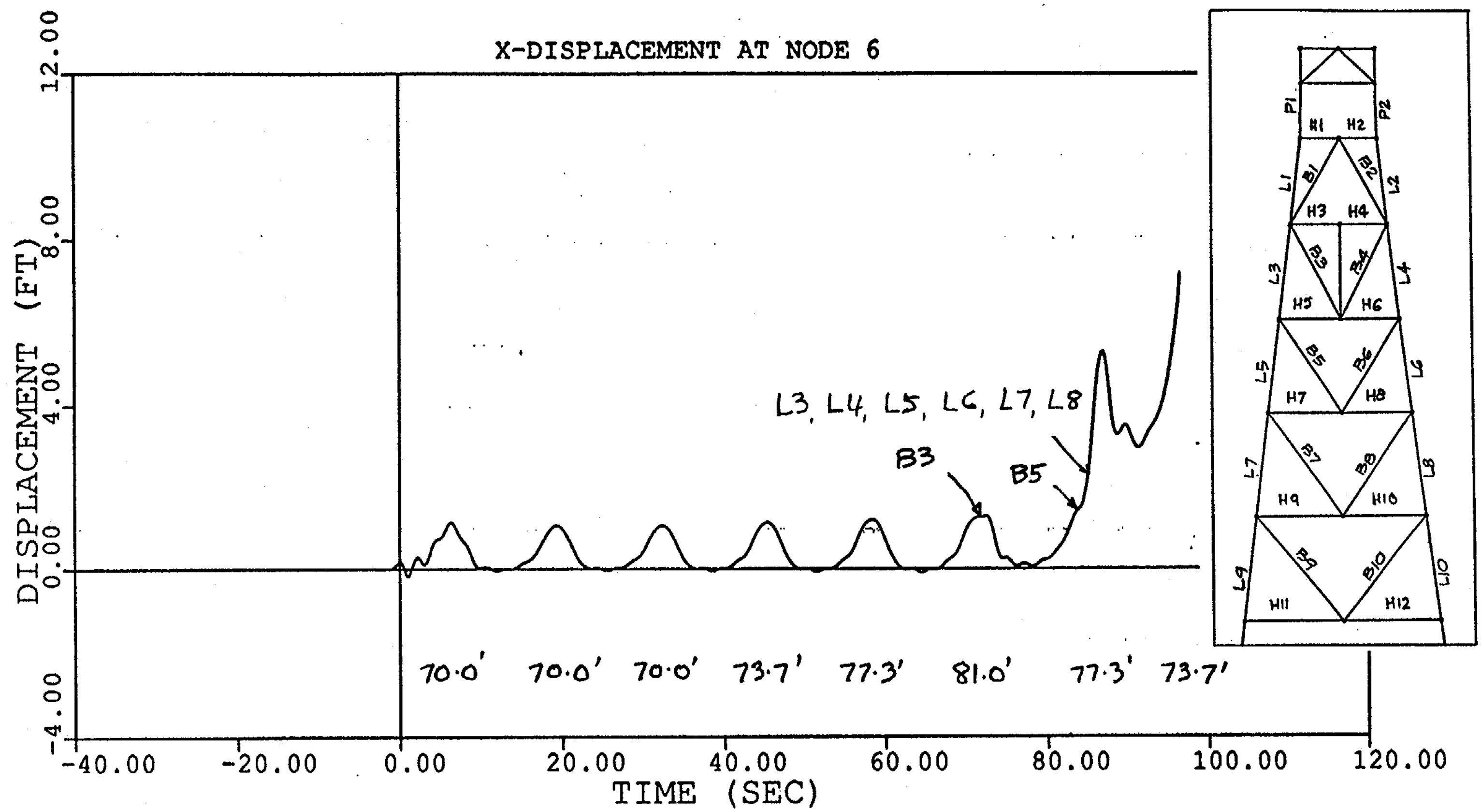
2-D. NO DECK LOAD. SYNTHETIC IRREGULAR WAVE 70-80-70

DISPLACEMENTS AT DECK

DATE - 07/28/93 SEAPOST Version 3.10 TIME - 11:03:43

Fig. 7-8

PMB Engineering Inc., San Francisco - Houston



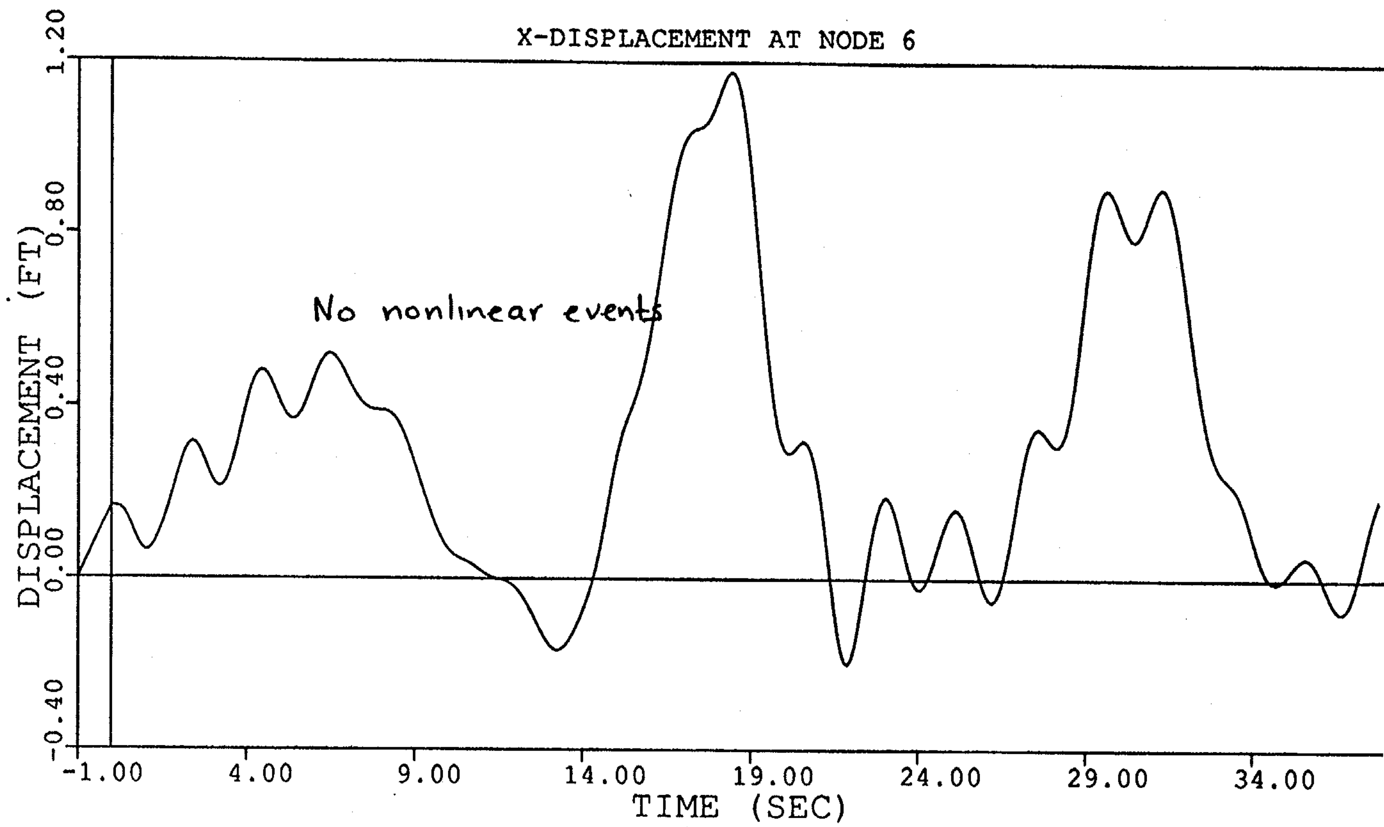
2-D. NO DECK LOAD. SYNTHETIC IRREGULAR WAVE 70-81-70

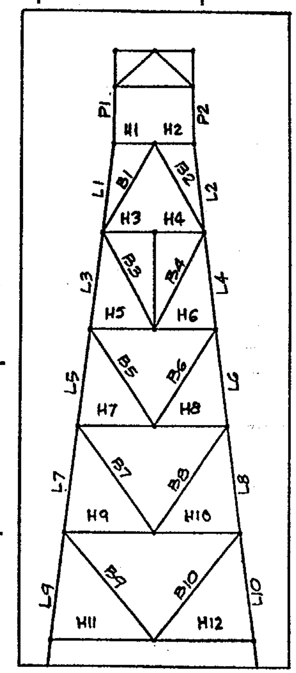
DISPLACEMENTS AT DECK

DATE - 07/28/93 SEAPOST Version 3.10 TIME - 14:53:20

Fig. 7-9

PMB Engineering Inc., San Francisco - Houston





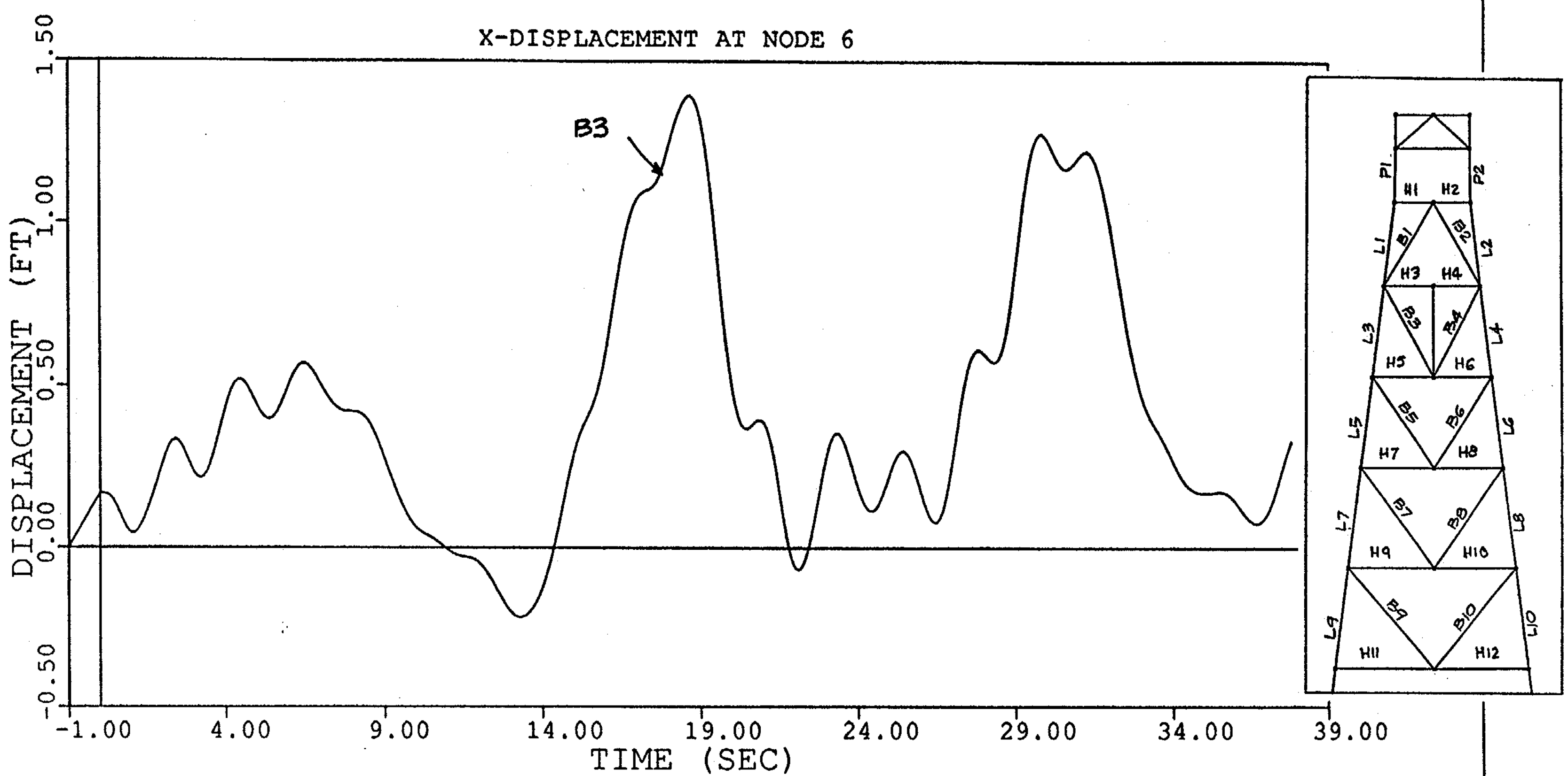
2D. NO DECK WAVE LOAD. MEASURED WAVE N1. 62.3 FT

DISPLACEMENTS AT DECK

DATE - 05/26/93 SEAPOST Version 3.10 TIME - 14:41:12

Fig. 7-10

PMB Engineering Inc., San Francisco - Houston



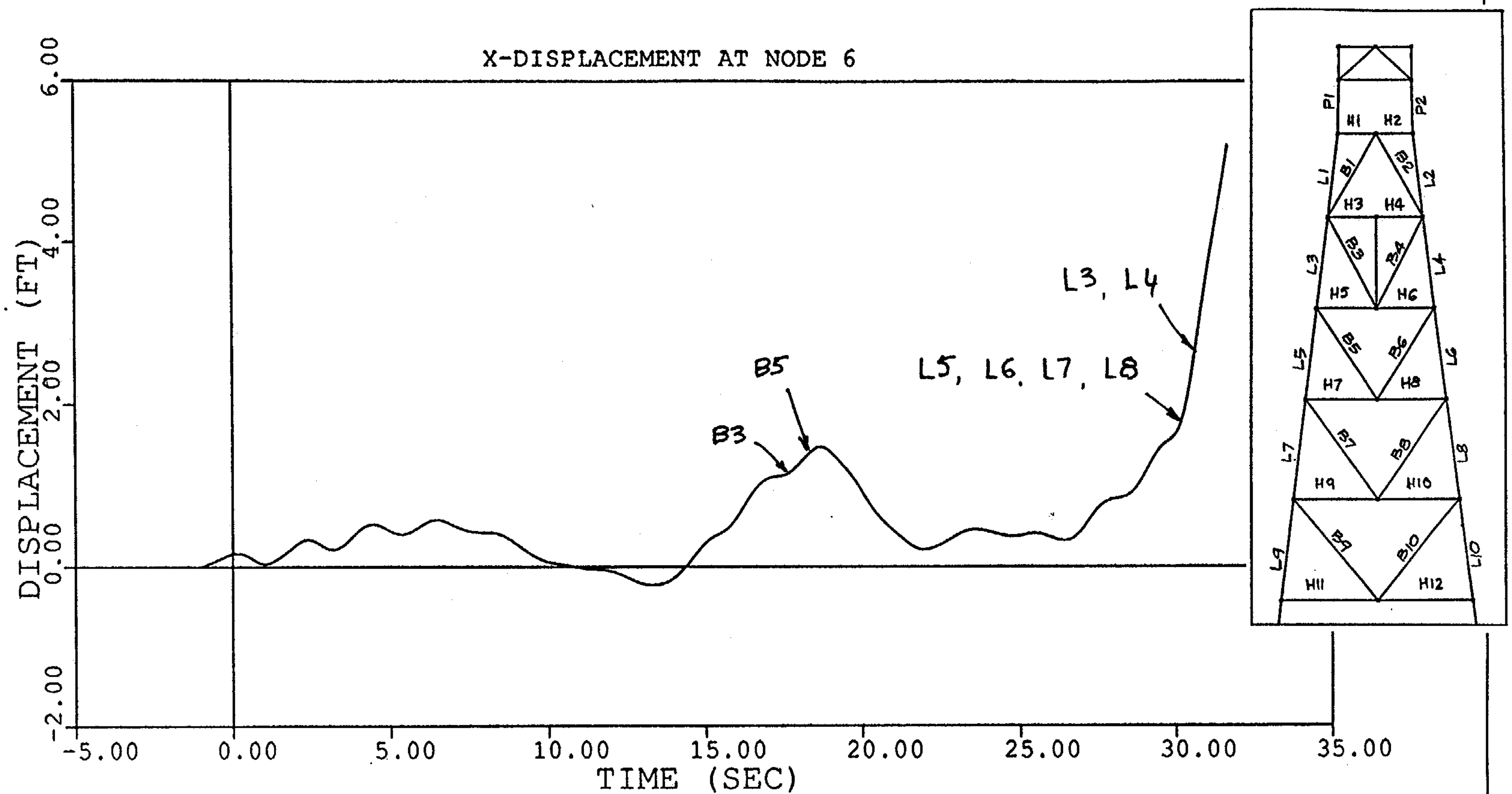
2D. NO DECK WAVE LOAD. MEASURED WAVE N1. 68.0 FT

DISPLACEMENTS AT DECK

DATE - 05/26/93 SEAPOST Version 3.10 TIME - 17:43:26

Fig. 7-11

PMB Engineering Inc., San Francisco - Houston



2D. NO DECK WAVE LOAD. MEASURED WAVE N1. 69.0 FT

DISPLACEMENTS AT DECK

DATE - 05/27/93 SEAPOST Version 3.10 TIME - 10:23:45

Fig. 7-12

Wave in Sequence m	H_m/H_1 (Stewart)	Multiple Waves 70ft-80ft-70ft	
		H	Hm/H_1
1	1.00	80.00	1.00
2	0.95	76.67	0.96
3	0.92	73.33	0.92
4	0.90	70.00	0.88
5	0.89	70.00	0.88

**Ordered Wave Height (Stewart)
and as Used in Multiple Wave Run**

Table 7-1

PERIOD	HEIGHT	PHASE
143.000000	0.0371565	179.99904
71.500000	0.0679809	0.00103
47.666667	0.0229805	179.99554
35.750000	0.0082965	0.01590
28.600000	0.0989276	179.99841
23.833333	0.1335565	0.00134
20.428571	0.0198482	179.99022
17.875000	0.1228692	0.00166
15.888889	0.8954223	179.99977
14.300000	2.0587065	0.00010
13.000000	70.3176955	180.00000
11.916667	2.0599230	0.00009
11.000000	0.8971714	179.99982
10.214286	0.1238966	0.00114
9.533333	0.0204729	179.99450
8.937500	0.1379421	0.00059
8.411765	0.1068616	179.99956
7.944444	0.0099509	0.00120
7.526316	0.0372746	-179.99936
7.150000	0.1832354	-0.00032
6.809524	0.3329817	-179.99972
6.500000	10.5372119	-0.00001
6.217391	0.3278546	-179.99954
5.958333	0.1607920	-0.00107
5.720000	0.0249476	-179.99240
5.500000	0.0044210	-0.04558
5.296296	0.0367570	-179.99435
5.107143	0.0309351	-0.00671
4.931034	0.0024374	-179.91733
4.766667	0.0101726	-0.01863
4.612903	0.0480592	-179.99642
4.468750	0.0759800	-0.00197
4.333333	2.2412117	-179.99995
4.205882	0.0736405	-0.00125
4.085714	0.0406812	-179.99856
3.972222	0.0067164	-0.00354
3.864865	0.0026950	179.99559
3.763158	0.0164594	0.00287
3.666667	0.0143693	179.99436
3.575000	0.0018391	0.06125
3.487805	0.0051214	179.97249
3.404762	0.0189392	0.00871
3.325581	0.0238452	179.99227
3.250000	0.6115135	0.00032
3.177778	0.0251507	179.99180
3.108696	0.0176264	0.01182
3.042553	0.0031341	179.93485
2.979167	0.0011224	0.17299
2.918367	0.0096040	179.98142
2.860000	0.0078629	0.02003
2.803922	0.0004722	0.27932
2.750000	0.0038212	0.02681
2.698113	0.0139910	179.99500
2.648148	0.0124054	0.00286
2.600000	0.1638200	-180.00000

SYNTHETIC IRREGULAR WAVES, 70-80-70
WAVE COMPONENTS

Table 7-2

SUMMARY OF RESULTS :	B A S E Case	Synthetic 70'-80'70'	Measured N1	Measured N2	Measured W1	Measured W2
1-st Period (sec)	2.08	2.08	2.08	2.08	2.08	2.08
STATIC PUSHOVER						
Crest Elevation	47.5	47.5	47.5	47.5	47.5	47.5
Fstat (Total)	1390	1390	1390	1390	1390	1390
Failure Mode *	J	P, J	P, J	P, J	P, J	P, J
DYNAMIC WAVES						
1) 1-st Event :						
Crest Elevation	45.8		68.0	60.0	49.9	44.4
Fdyn (Total)	1330	Not	1320	1260	1340	1280
Failure Mode *	J		J	J	J	J
2) Collapse :						
Crest Elevation	46.5	46.5	69.0	62.0	51.3	45.7
Fdyn (Total)	1350	1380	1344	1400	1390	1330
Fdyn/Fstat	0.97	0.99	0.97	1.01	1.00	0.96
Failure Mode *	J	J	J	J, P	J, P	J

Table 7 - 3. Irregular Waves

* J = Jacket
P = Portal

Section 8
Modified Platform C

8.1 OBJECTIVES

A study on a single platform, even if it has typical configuration and loading, could lead to conclusions that may not be valid for other platforms. For this reason a number of variations on the basic platform geometry and loading were made, and the static and dynamic collapse loads were determined for each.

The purpose of these studies was not to test a range of real structures, but to include variations in the structures that might indicate sensitivities to variations in mass, weight, strength stiffness and configuration. For instance, by bracing the legs below the deck, the failure mechanism could be changed from collapse of the portal to a failure in the jacket itself. This change could lead to rather different conclusions regarding the importance of dynamics.

8.2 APPROACH

The structure used in the studies described here was the two-dimensional structure described in Section 4.5.

In all the variations described in this section, wind, wave and current loads in the deck were included in the applied loads, along with current and wave loads on the jacket. Deck wave loads were computed by the preliminary API method in all cases. Deck waves were the focus of the investigation since it was thought that structures might be more sensitive to dynamics if there were loads in the deck.

For each modification the procedure was the same. A static pushover was performed as described in Section 5.0, to determine the static capacity. Then regular waves somewhat smaller than those that caused static collapse were run by the platform and the response of the platform was observed. The height of these waves was then increased in a series of trials until collapse of the platform occurred, defined by deck deflections increasing rapidly from about 3 or 4 times the deck deflection before failure started.

The next sections describe these modifications. No detailed descriptions of the responses are generally deemed necessary, since they follow generally the same characteristics as described in Section 6.2. The results from the studies are summarized in Tables 8-1, 8-2, and 8-3. Descriptions of the entries in this table are given in Section 6.3, relating to Table 6-1.

8.3 BRACING CONFIGURATION CHANGES

The base case configuration, as shown in Figure 4-12, has K-braces in the jacket and an unbraced portal frame below the deck.

The following modifications were tested.

1. Diagonal braces replacing the K-braces. The existing braces sizes would have resulted in members much too slender if called upon to act as diagonals. They were therefore increased in size so that their slenderness ratio were 70 to 75, similar to diagonal braces in the longitudinal direction. The new configuration is shown in Figure 8-1 with the sizes of the new members.

2. Cross braces replacing the K-braces. With cross braces effectively connected at the mid-points, the slenderness ratio is rather less than for K-braces. Figure 8-2 shows the configuration and member sizes.

3. Cross braces, with doubled mass. Since cross bracing the structure stiffened it somewhat, a variation was included in which the jacket and deck mass and weight were doubled. Cross bracing was as described previously and shown in Figure 8-2.

4. Fully diagonally braced jacket. A case was included in which the portal frame was also diagonally braced. In addition, this bracing decreases the first natural period of the platform. The member was sized to give slenderness ratio similar to existing diagonal braces. The configuration and member size is shown in Figure 8-3. Since failure occurred in a brace for the condition without the portal braced, this addition of portal bracing had little effect.

The results of this study are shown in Table 8-1. Description of the entries is given in Section 6.3. It is seen that there is a slight increase in the capacity when simple one degree of freedom dynamics from the wave period is included. If this is excluded the dynamic capacity is within 4% the same as the static capacity.

Generally, the investigation concluded that these bracing configuration changes did not significantly vary from the results for the K-brace configuration (i.e., static pushover is a good estimate of platform capacity).

8.4 MASS AND WEIGHT CHANGES

1. Jacket mass and weight multiplied by 3. To test the effect of changes in the natural periods and leg strength of the structure, the mass and weight of the jacket

below the deck was tripled. The mass of the deck and equipment was kept the same as the base case.

2. Jacket mass and weight multiplied by 3, with damping multiplied by 1.5. This case was the same as the previous one except for increase in damping.

3. Jacket mass and weight multiplied by 5. To test the effect of greater changes in the natural periods and weaker legs, the mass of the jacket below the deck was multiplied by 5.

4. Deck mass and weight multiplied by 2. Since the first natural period of the platform is more sensitive to deck mass than jacket mass, the first variation of deck mass was made by doubling it, to get natural period variation generally similar to that from tripling the jacket mass. This will also weaken the legs somewhat. It was expected that, by varying the deck mass and not the jacket, for a given first natural period there might be significant differences in portal response between this variation and the variation including changes in jacket mass and weight.

5. Deck mass and weight multiplied by 2. Transient Study. There was some concern that the effect of transients from the initial conditions adopted might be more important for this case than for the base case, since the period had changed significantly. For this reason a study was included in which the water particle kinematics from waves were ramped up linearly from zero at time zero, to full values after three wave periods. After this full wave loads were applied and the analysis was run for a further two wave periods. This process reduces the initial transients greatly. The deck displacement history for the 77.0 ft wave is shown in Figure 8-4. It was found that the effects of transients was not great, the structure collapsing at the wave height as for the previous case.

6. Deck mass and weight multiplied by 3. This is a more extreme variation of that described in 4) above.

7. Deck mass and weight multiplied by 3. No deck wave forces. Case 6 results showed significant difference between collapse loads from static and from dynamic analyses. It was thought important to see whether this was also true when there was no wave in the deck.

Table 8-2 summarizes the results of this part of the study. Descriptions of entries are given in Section 6.3.

It can be seen that dynamic capacities are always close to or exceeding the corresponding static capacities. In fact wave heights associated with static and dynamic collapses are all within 3 ft of each other.

The greatest difference occurs with the increased deck mass cases, in which multipliers of 2 and 3 were used. A multiplier of two already represents a rather heavy deck for this type of structure. The difference between dynamic and static capacities for these cases is 12%, which is amplified to 20% if the dynamic amplification factor associated with the wave period and first structural period is used.

When the wave is not in the deck, the difference between static and dynamic strength is greatly reduced, and for this case there is no significant difference.

Thus, within the limits of this part of the study, it appears that with rather heavy structures or decks, and when wave deck loads are considered, up to about 12% conservatism can occur if static pushover is used as a measure of dynamic strength.

When deck wave loads are not considered, static pushover gives an excellent estimate of dynamic platform strength.

8.5 STIFFNESS AND LEG STRENGTH CHANGES

1. Reduction of leg and pile thicknesses with a factor of 0.75. The effect of this was a small increase in the first period of the structure, but more importantly, a reduction in the strength of the jacket.

2. Reduction of leg and pile thicknesses with a factor of 0.50. This is an extension of the changes in the previous case.

3. Reduction of the thickness of the legs in the portal with a factor of 0.75. This reduced the strength of the portal, leaving the rest of the jacket intact.

4. Reduction of the thickness of the legs in the portal with a factor of 0.50. This is a extension of the changes in the previous case.

5. Reduction of the elastic modulus of all members by a factor of 0.50. This had the effect of raising the period of the jacket, and lowering the strength greatly, due mainly to the reduced buckling capacity of the braces.

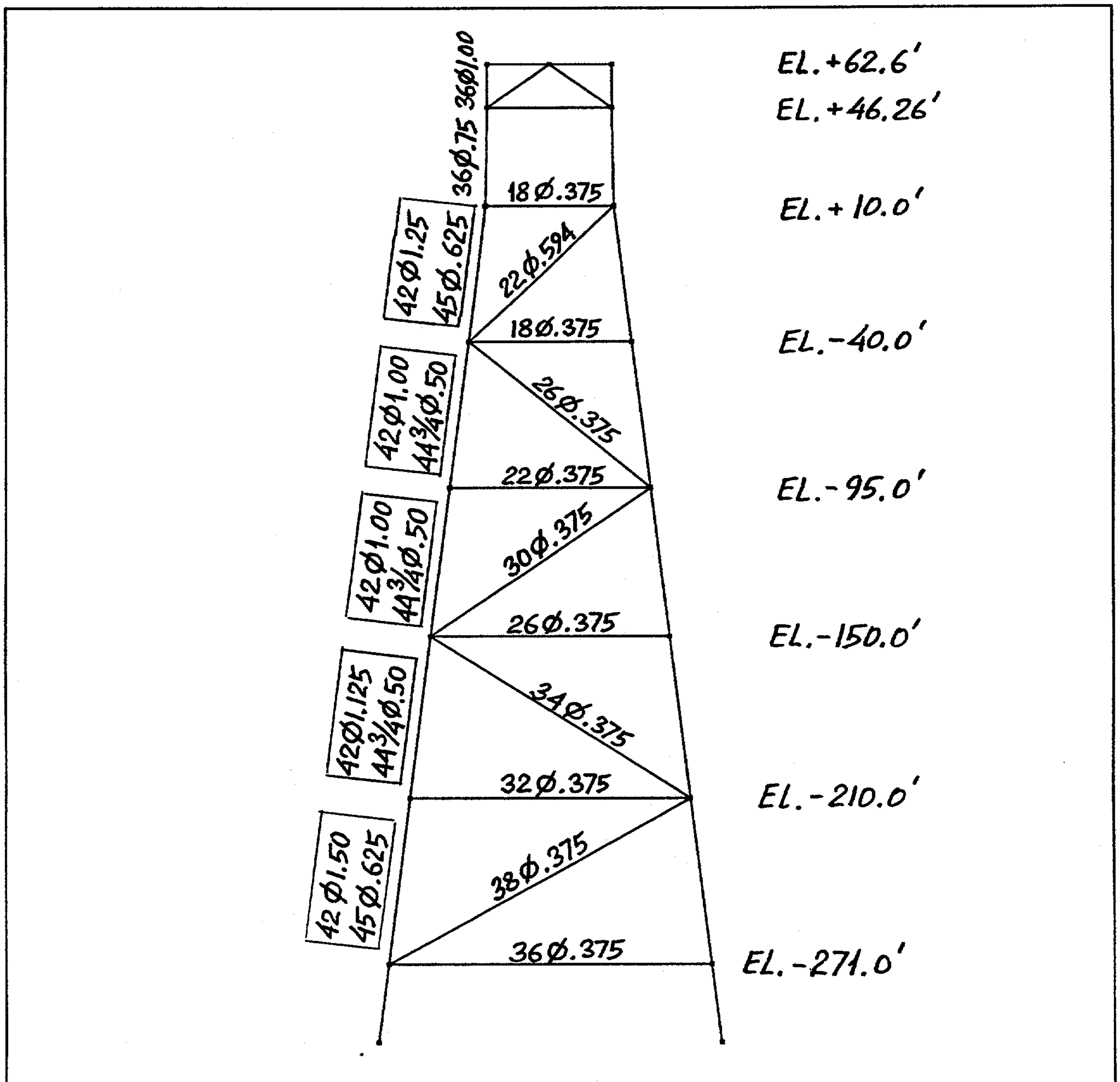
6. Ungrouting the piles. The piles were treated as fixed to the jacket at the top of the battered section of the jacket legs, and free to slide inside the legs below this

level. This lead to significant increase in first period of the jacket, member strengths being unaffected.

7. Increase in damping by 1.5 times. This is not a stiffness change, but it is included in this group of tests for want of a better place to put it.

The results of this study are shown in Table 8-3. Description of the entries is given in Section 6.3. It is seen that there is some increase in the capacity when dynamics is considered. The variations considered that showed the greatest differences are for rather extreme variations. In all cases the waves causing dynamics and static collapse were with 3.5 ft.

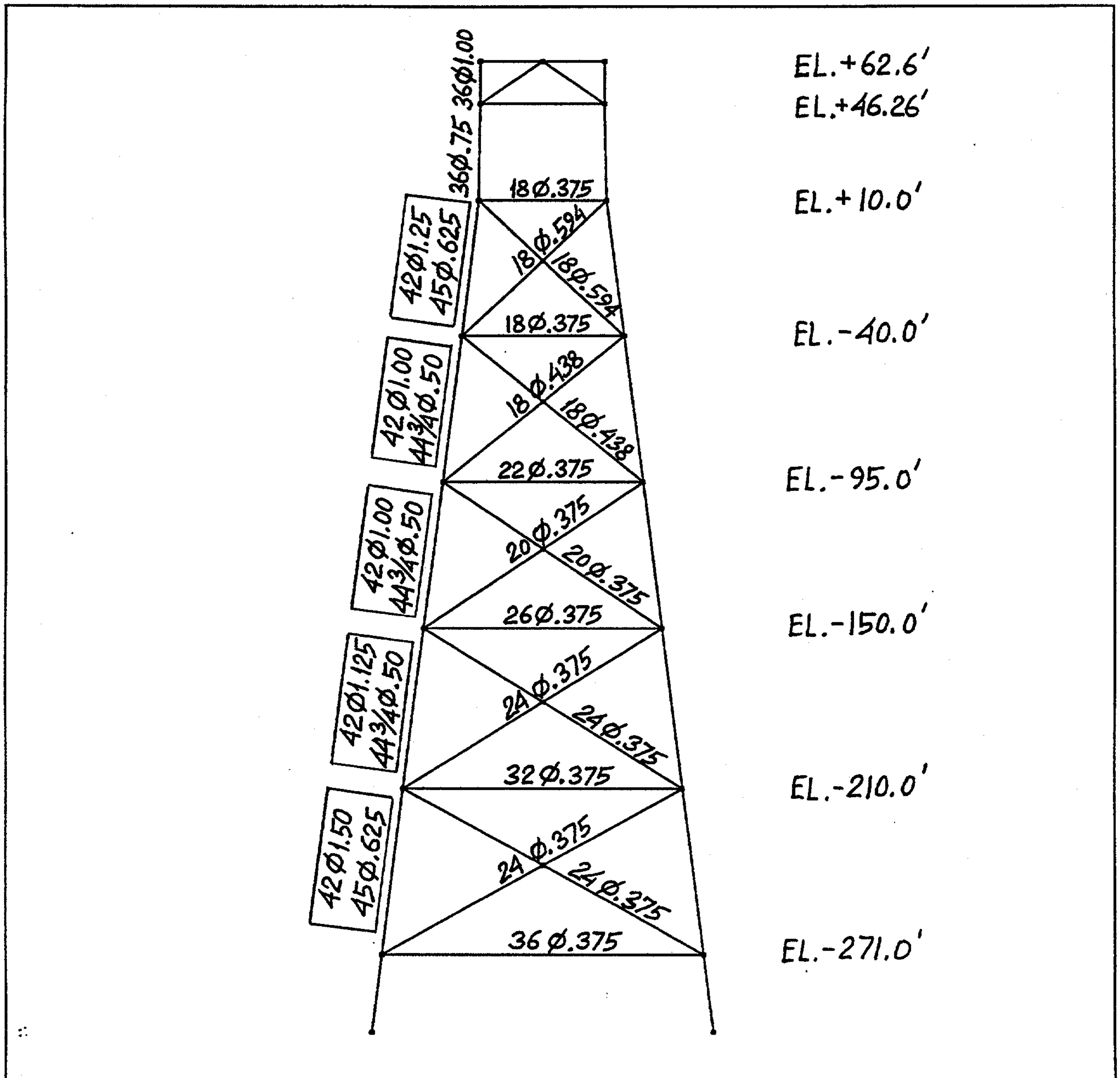
It can be concluded that static capacity is a good estimate of dynamic capacity, erring on the conservative side when differences are up to 15%.



CAP z x 2-D Model with Diagonal Braces. Member Sizes

Project: DiagBraces Model: InDeck74 Version: 1

Figure 8 - 1

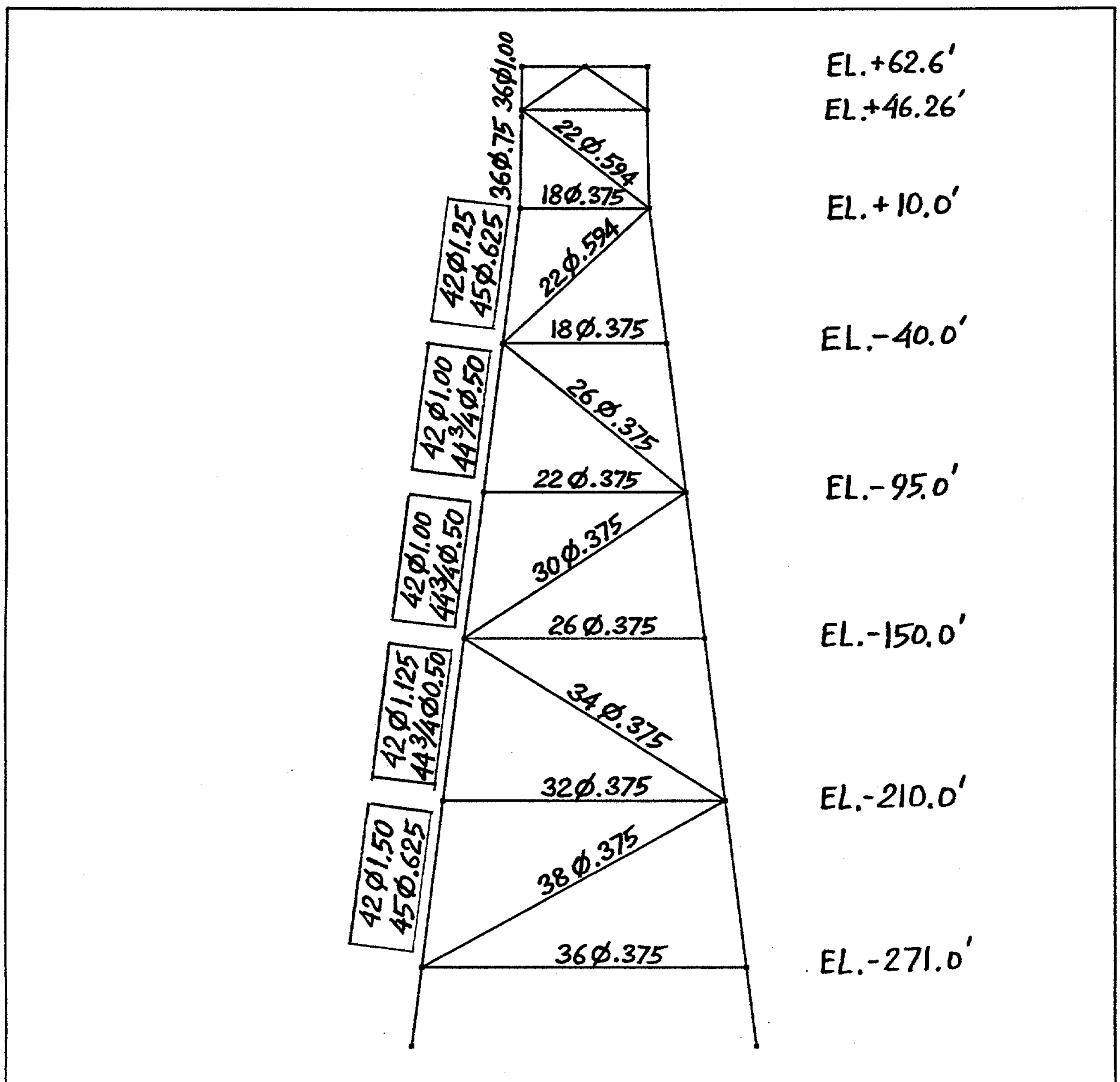


CAP Z X

2-D Model with X-Braces. Member Sizes

Project: XBraces Model: StatPushover83 Version: 1

Figure 8 - 2



CAP z x 2-D Model w/Diagonal Braces & Portal Brace

Project: DiagBrPortal Model: InDeck74 Version: 1

Figure 8 - 3

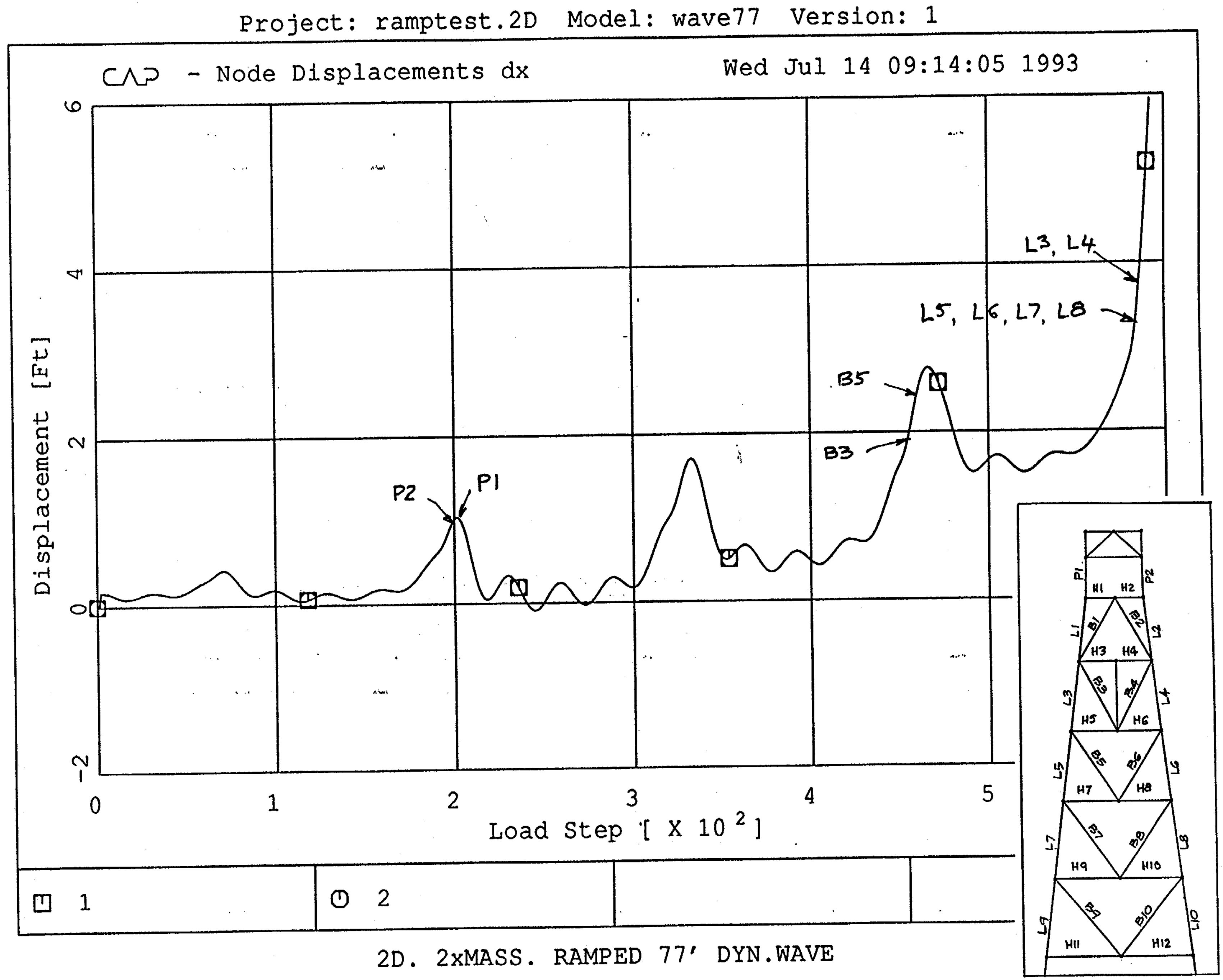

Project: ramptest.2D Model: wave77 Version: 1
CAP - Node Displacements dx
Wed Jul 14 09:14:05 1993
Displacement [Ft]
6
4
2
0
-2
0
1
2
3
4
5
Load Step [X 10 2]
P2
P1
B5
B3
L3, L4
L5, L6, L7, L8
1
2
2D. 2xMASS. RAMPED 77' DYN.WAVE
P1
P2
H1
H2
L1
B1
B2
L2
H3
H4
B3
B4
L3
L4
H5
H6
B5
B6
L5
L6
H7
H8
B7
B8
L7
L8
H9
H10
B9
B10
L9
L10
H11
H12

Fig. 8-4

SUMMARY OF RESULTS :	BASE	Diagonal Braces	Cross Braces	Mass x 2, Cross Braces	Fully Diagon. Braced
1-st Period (sec)	2.08	2.04	1.97	2.71	1.67
STATIC PUSHOVER					
Wave Height	74.5	72.0	75.0	73.5	72.0
Fstat (Total)	1400	1230	1390	1310	1230
Failure Mode *	P, J	J, P	P	P	J
DYNAMIC WAVES					
1) 1-st Event :					
Wave Height	72.0	70.5	74.5	71.5	70.5
Fdyn (Total)	1280	1150	1360	1210	1150
Failure Mode *	P	J	P	P	J
2) Collapse :					
Wave Height	75.0	74.0	76.0	74.5	74.0
Fdyn (Total)	1430	1350	1450	1360	1350
Fdyn/Fstat	1.02	1.10	1.04	1.04	1.10
Failure Mode *	J, P	J	P	P	J

Table 8 - 1. Bracing Configuration Modifications

* P = Portal
J = Jacket
P,J = Portal then Jacket
J,P = Jacket then Portal

SUMMARY OF RESULTS :	B A S E Case	Triple Jacket Mass	Triple Jack.Mass, Damp x 1.5	Jacket Mass x 5.0	Double Deck Mass	Double Deck Mass Ramped	Triple Deck Mass	No Deck Wave; Triple Deck Mass
1-st Period (sec)	2.08	2.30	2.30	2.53	2.79	2.79	3.35	3.35
STATIC PUSHOVER								
Wave Height	74.5	73.0	73.0	74.0	74.0	74.0	73.0	80.0
Fstat (Total)	1400	1330	1330	1370	1370	1370	1330	1380
Failure Mode *	P, J	J	J		P	P	P	J
DYNAMIC WAVES								
1) 1-st Event :								
Wave Height	72.0	73.5	74.5	74.5	74.0			73.0
Fdyn (Total)	1280	1350	1400	1400	1370	Not	Not	1210
Failure Mode	P	J	P, J	P, J	P			P
2) Collapse :								
Wave Height	75.0	74.5	75.0	75.0	77.0	77.0	76.0	78.0
Fdyn (Total)	1430	1400	1430	1430	1540	1540	1490	1330
Fdyn/Fstat	1.02	1.05	1.08	1.05	1.12	1.12	1.12	0.96
Failure Mode	J, P	P, J	P, J	P, J		P, J		P, J

Table 8 - 2. Mass and Weight Modifications

* J = Jacket
P = Portal
P,J = Portal then Jacket

SUMMARY OF RESULTS :	B A S E	Ungrouted	Elastic Modulus x 0.5	All Thickness x 0.75	All Thickness x 0.50	Portal Thickness x 0.75	Portal Thickness x 0.50	B A S E w/damping x 1.50
1-st Period (sec)	2.08	2.62	2.95	2.27	2.62	2.16	2.32	2.08
STATIC PUSHOVER								
Wave Height	74.5	74.0	57.5	72.5	70.0	72.5	69.5	74.5
Fstat (Total)	1400	1370	860	1300	1180	1300	1160	1400
Failure Mode *	P, J	P, J	J	P	P	P	P, J	P, J
DYNAMIC WAVES								
1) 1-st Event :								
Wave Height	72.0	72.0	56.0	71.0	67.0	72.0	68.0	72.0
Fdyn (Total)	1280	1280	830	1230	1060	1280	1090	1280
Failure Mode *	P	P	J	P	P	P	P	P
2) Collapse :								
Wave Height	75.0	75.0	58.0	75.0	72.0	76.0	72.0	75.0
Fdyn (Total)	1430	1430	870	1430	1280	1460	1280	1430
Fdyn/Fstat	1.02	1.04	1.01	1.10	1.08	1.15	1.10	1.02
Failure Mode *	J, P	J, P	J	P, J	P	P, J	P	J, P

Table 8 - 3. Stiffness and Strength Modifications

* P = Portal
J = Jacket
P.J = Portal then Jacket
J.P = Jacket then Portal

Section 9
Dynamic Effects Observations

9.1 DEFINITION OF TERMS

In considering dynamic effects in the prediction of the wave return period, two types of effects need to be distinguished: *Dynamic Effect Type I* will refer to the inertia forces generated from accelerating masses and *Dynamic Effect Type II* will refer to the cyclic or hysteretic loading path imposed on members in a time domain environment in contrast to the monotonic loading which accompanies static pushover (see Figure 9-1).

For the platform configurations studied in the project, both types of dynamic effects were evident, however the net effect was that static pushover adequately identified the critical wave height.

Before proceeding to highlight the particular dynamic behaviors observed in the study, it will be useful to also summarize the terms that are commonly associated with characterization of the static pushover signature of a structure. Figure 9-2 defines four important measures of the structure based on the static pushover load-deflection curve. The *ultimate capacity*, R_{ult}, is of course used to identify the critical wave height. The *robustness factor*, can be used to assess the ability of the structure to handle subsequent waves after the critical wave. The *ductility factor*, essentially defines when the structure deformation will approach the development of the final collapse mechanism, which this project defines as imposing a portal mechanism into the legs or vertical load carrying system. The *redundancy factor*, RF, indicates the reserve capacity between the initial member failure and ultimate capacity.

Figure 9-3 shows two examples of how these terms are defined. The figure shows a static pushover of a K-braced and X-braced platform, including load deformation after the structures have reached ultimate capacity. The K-braced structure has both first member failure and ultimate capacity of 1400 kips, followed by a post-ultimate strength of 500 kips. The resulting response parameters are: Redundancy Factor = 1.0; Robustness Factor = 0.36; and Ductility Factor = 1.9/1.3 = 1.46. The X-braced structure has a first member failure at 1950 kips and an ultimate strength of 800 kips. The resulting response parameters are: Redundancy Factor = 1.23; Robustness Factor = 0.33; and Ductility Factor = 2.4/2.1 = 1.14. These results indicate that the X-braced platform has considerably more redundancy and can sustain some limited damage prior to collapse when compared to the K-braced platform. However, both structures have about the same robustness and ductility factors due primarily to the development of hinges in the legs (which are similar in the platforms) following failure of the bracing system.

9.2 OBSERVED DYNAMIC EFFECT TYPE I

Two significant dynamic effects were observed in the study. The first was an inertia force or Type I effect associated with wave loading in the deck. The impulsive load of the wave hitting the deck resulted in a DAF as large as 1.7 for the shear force imposed on the deck portal columns connecting to the top of jacket. A 1/2 sine wave impulse of 1 second duration and a portal natural period of 1 second directly translate into a DAF of 1.7 as seen in the shock spectrum of Figure 9-4. The dynamic time history response of the structure agreed well with the shock spectrum. Figure 9-5 compares the portal shear with the applied deck wave load. When the portal was forced to remain linear, the portal shear was amplified to 1.7 times the applied wave load, consistent with the shock spectrum. For the study structure the strength of the portal frame was close to the static applied load level. When the nonlinear analysis of the structure was performed, as the portal shear began to amplify, it was limited by the leg strength, as seen in Figure 9-5. For this particular portal, the loading reversed before collapse deformations could build up.

The lesson learned from this observation is that a straight application of the static pushover in which the applied load pattern was strictly representative of the static wave force would not have identified the potential for damage in the portal frame. One needs to perhaps scale the deck region of the static push pattern by an appropriate DAF established from the shock spectrum (see Figure 9-6).

9.3 OBSERVED DYNAMIC EFFECT TYPE II

The second significant dynamic event observed was of the Type II effect, hysteretic behavior. In every case analyzed in the study, the structure did not attain collapse deformations during the application of the 1st large "critical" wave in the time trace. Failure of the structure occurred during the wave cycles following the critical wave. Typically the first large wave would damage a principal brace but the load would reduce and reverse direction before collapse deformations could build. On the subsequent wave cycle the damaged structure would try to resist the load in the remaining portal frame which would subsequently develop leg hinges and a portal collapse mechanism. The reason the structure is weak in the second cycle is due to the one way nature of wave plus current loading and the nature of the hysteretic behavior of tubular braces. Figure 9-7 shows a representative force-deformation curve for one of the tubular braces in the example structure. On the critical wave cycle the brace buckles (point a) and proceeds to a post buckling deformation point b, at which time the wave load reverses and begins to unload the brace. However, because of the predominately one way loading nature of wave plus current, the reverse load does not fully tension and straighten (and consequently heal) the brace (point c). Thus at the start of the next wave cycle after the critical wave, the buckling strength is greatly reduced (point b vs.

point a). The structure is now significantly softer as the second wave force builds, permitting the larger portal deformations to build and lead to collapse.

Based on this observation one could postulate that if the structure were hit by a freak wave followed by a very small wave train, it could survive. For GOM storm wave traces this is not the norm. Instead, wave ranking has the second and third highest waves within 95% of the largest wave. For the study structure which has a small robustness factor (0.4), the second and subsequent waves would have to be less than 60% of the critical wave height to avoid collapsing the damaged structure.

The robustness factor for the structure is a useful measure of the framing systems ability to handle successive load trains. For the study structure, the very low robustness of 0.4 implies that the second wave load would have to be less than 40% of the critical wave.

9.4 RECOMMENDED PROCEDURE FOR EVALUATING PLATFORM CAPACITY

Based on the findings of the study, the following procedure is recommended for establishing the overload capacity of a structure and the associated critical wave return period. The procedure will essentially default to a common static evaluation for most platforms, but will also identify any situations in which dynamics may significantly alter the evaluation. The most likely area for such deviation would be in the portal framing region just below the deck area.

1. Obtain the Static Resistance Characteristics of Platform

   - Perform Static Monotonic Pushover using Static Wave Load Pattern
   - Identify Resistance Parameters From Load-Deflection Curve
     - Ultimate Capacity
     - Redundancy Factor
     - Robustness Factor
     - Ductility Factor

2. Obtain Principal Dynamic Characteristics of the Platform

   - Platform Global Period
   - Portal Period

3. Determine Storm Wave Force Ordering for Platform Site

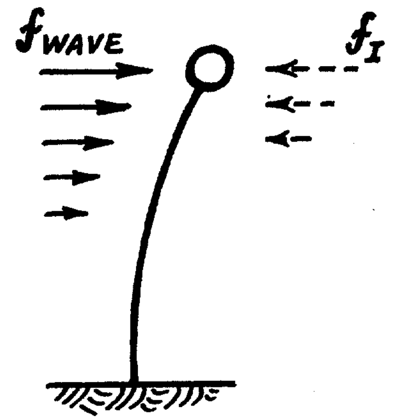
4. Based on Above, Modify Static Pushover Pattern by DAF
   (Dynamic Effects Type I)

   - Global (Classical SDOF DAF)
   - Portal (Use Shock Spectra DAF)

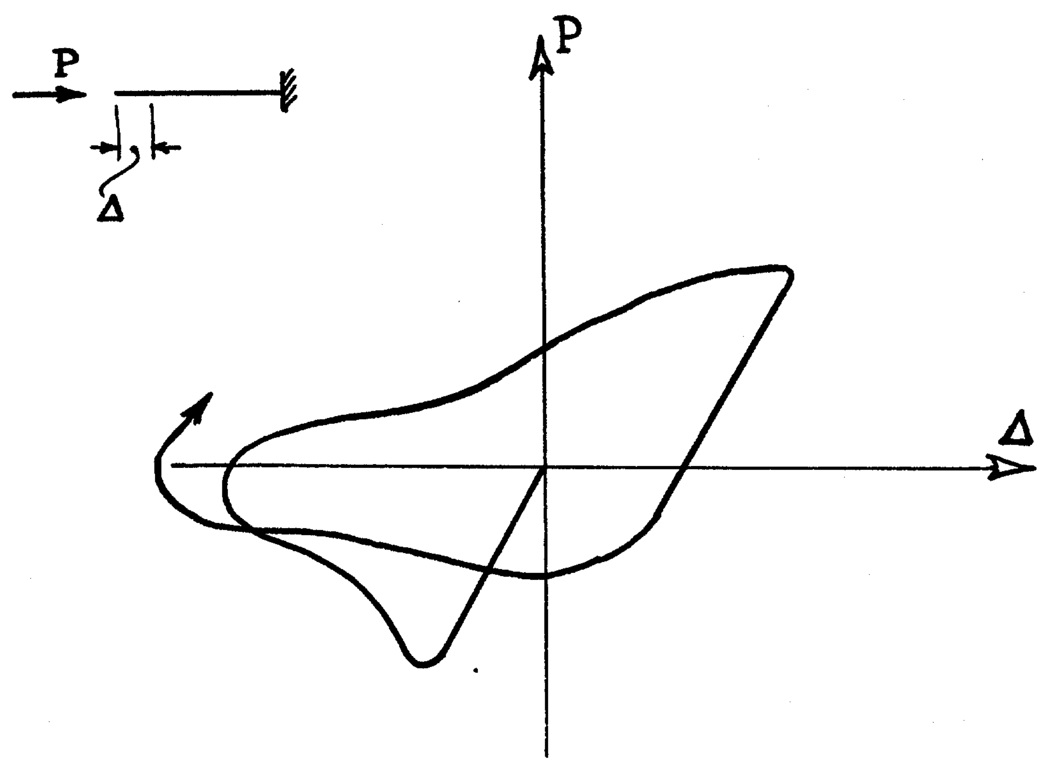
5. Consider Robustness Factor and Wave Force Ordering
   (Dynamic Effects Type II)

   - If robustness >80-90%, potential for survivability in GOM is significant.

• TYPE 1, INERTIA & DAMPING FORCES

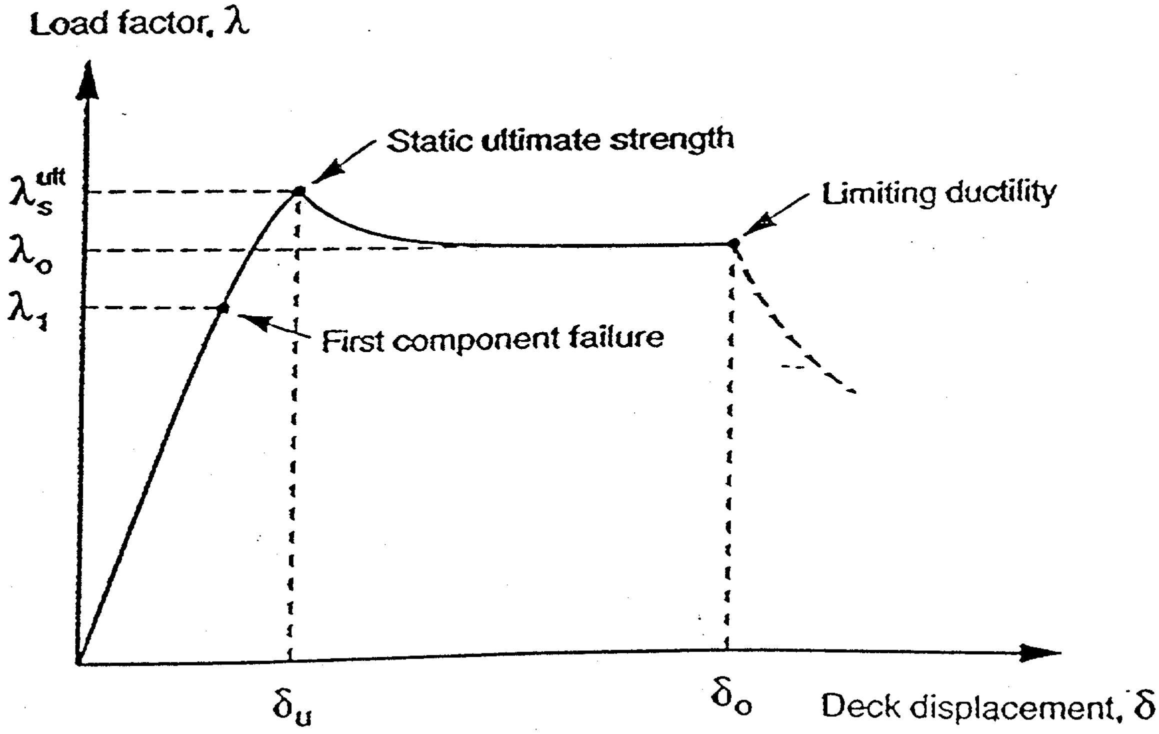


• TYPE 2, CYCLIC BEHAVIOR



DEFINITION OF DYNAMIC EFFECTS

Fig. 9 - 1



$$RF = \frac{\lambda_S^{ult}}{\lambda_1}$$ Redundancy Factor

$$\alpha = \frac{\lambda_0}{\lambda_S^{ult}}$$ Robustness Factor

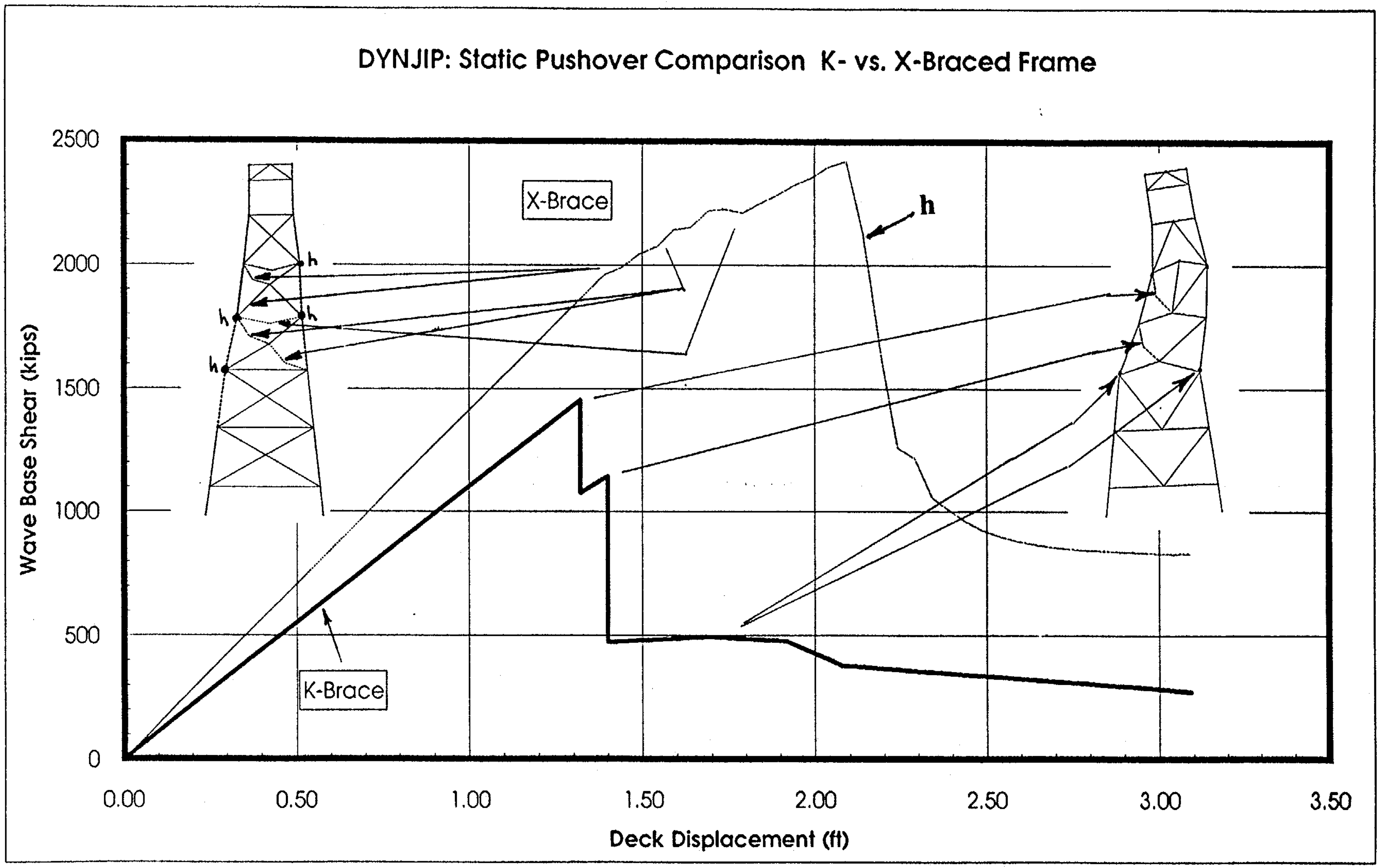
$$\mu_{max} = \frac{\delta_0}{\delta_u}$$ Ductility Factor

Pushover Response Parameters

(G. Stewart)

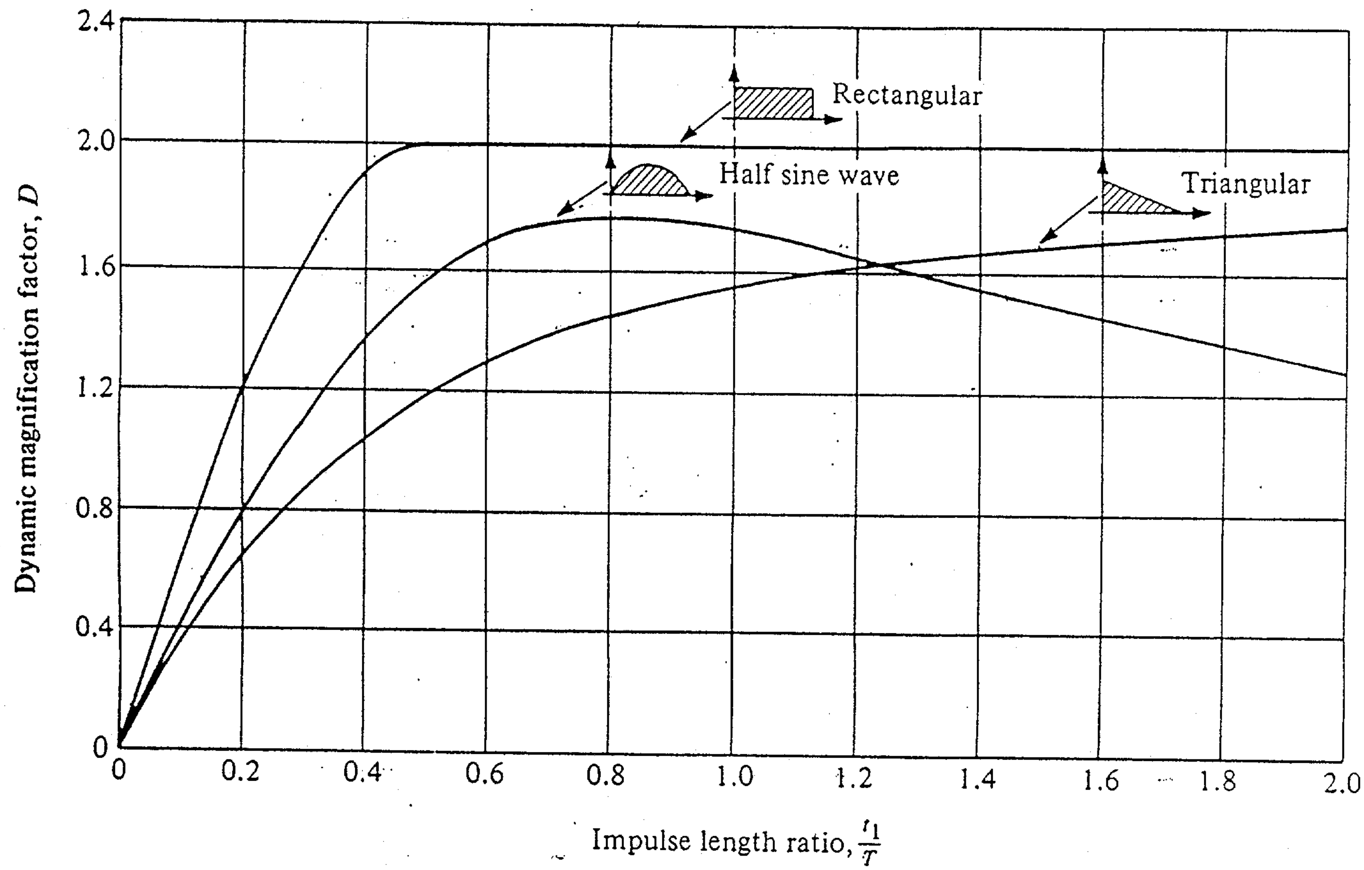
DEFINITION OF STATIC PUSHOVER PARAMETERS

Fig. 9-2



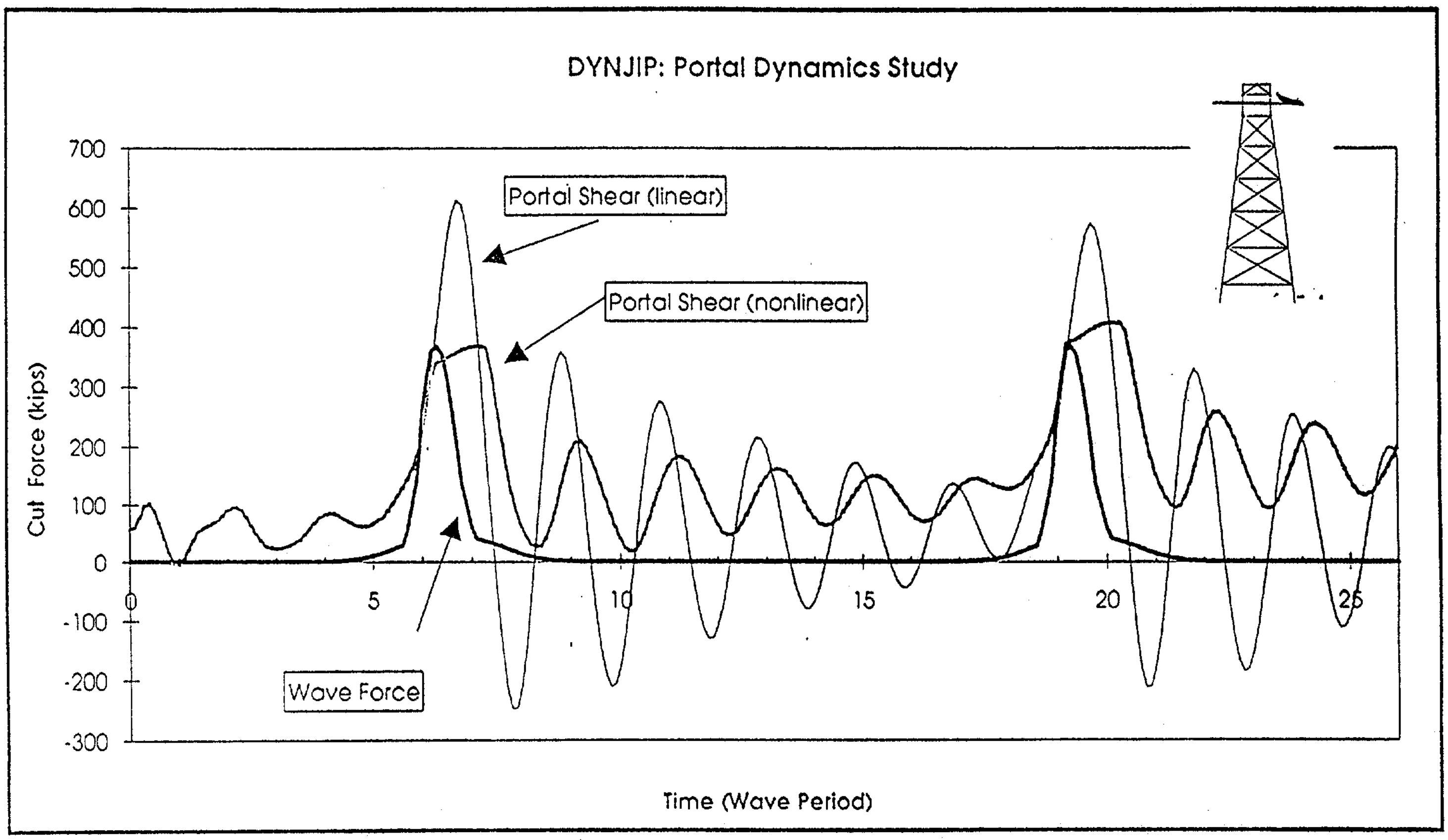
DYNJIP: Static Pushover Comparison K- vs. X-Braced Frame
Wave Base Shear (kips)
2500
2000
1500
1000
500
0
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
Deck Displacement (ft)
X-Brace
K-Brace
h

Fig. 9 - 3



SHOCK SPECTRUM

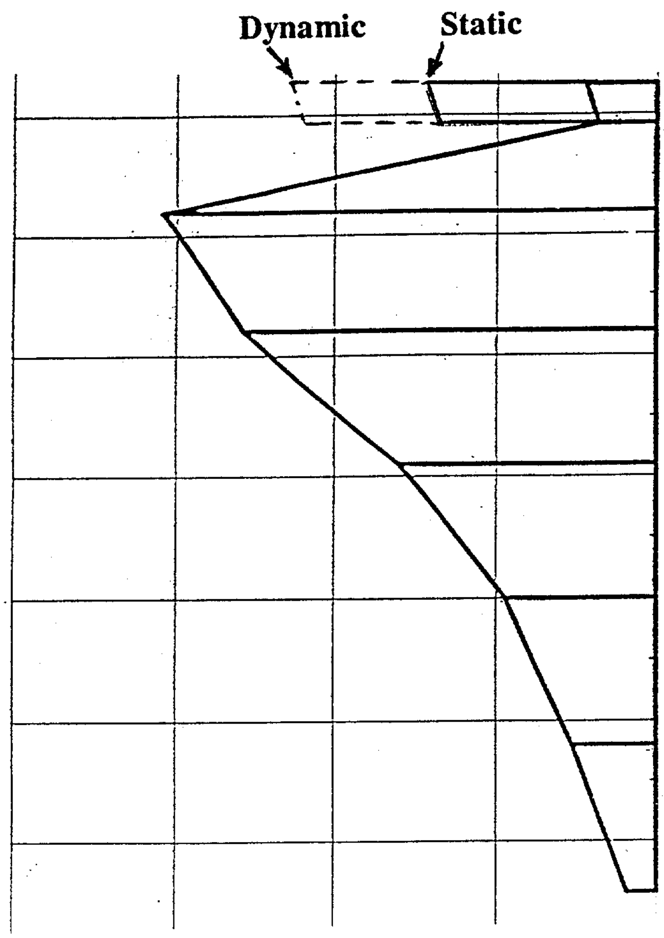
Fig. 9 - 4

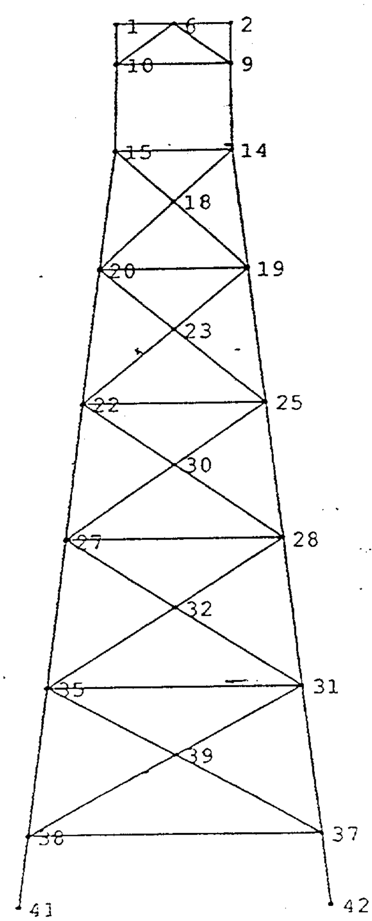


IMPULSE RESPONSE OF DECK PORTAL

Fig. 9 - 5

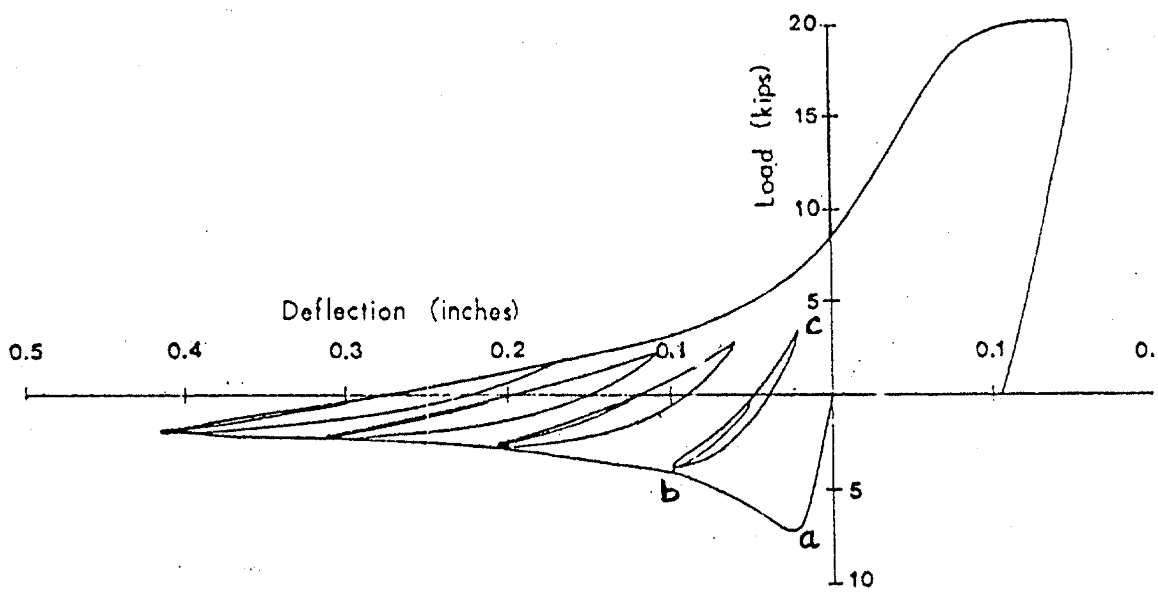
Jacket and Deck Wave Loads





MODIFIED STATIC PUSHOVER PATTERN

Fig. 9 - 6



KL/*r* = 120, Load Sequence 1 (1 in. = 25.4 mm; 1 kip = 4.45 kN)

TYPICAL TUBULAR BRACE CYCLIC LOAD-DEFLECTION CURVE

Fig. 9 - 7

Section 10
Conclusions and Recommendations

This study compared the collapse loads from waves when analyzed dynamically with that from a static pushover analysis. The basic platform was a typical 8-leg Gulf of Mexico jacket in 271 ft of water. Modifications to the structure configuration, member sizes, and deck and platform weights and masses were made. Regular and irregular waves were considered. Most of the analyses included waves in the deck, with deck wave loads computed according to the preliminary API RP 2A recommendations. Table 10-1 summarizes the results of this study.

10.1 CONCLUSIONS

Overall conclusions of the project are as follows:

- For the platform configurations studied in the project, both inertia force and cyclic loading dynamic effects were observed. Large inertia force amplification was observed in the portal frame for the impulsive deck load. Cyclic loading behavior accounted for the observation that the structure did not fail during the critical wave cycle, but in a subsequent smaller wave cycle. However, for the structural configurations studied these observed dynamic effects did not significantly change the critical wave height identified by the more conventional static pushover method.

- For exceptionally heavy decks and exceptionally weak structures (e.g., heavily corroded) where there is wave loading in the deck, the platform capacity determined by dynamic analysis was higher than that determined by the static pushover. The increase ranged from 10 to 15 percent. These situations (heavy deck or weak structure) may not be typical of Gulf of Mexico operations. This increase was not apparent for wave-below-deck loading conditions.

- Changes in configuration of the platform bracing scheme (k-brace versus diagonal-brace versus x-brace) had little effect on the above conclusions (i.e. all exhibit similar behavior).

10.2 RECOMMENDATIONS

Overall recommendations for the project are as follows:

- It is recommended that the industry standard static pushover still be used as a basis for estimating platform capacity. For cases where there is wave-in-the-deck, it is recommended that further investigation be taken to determine the potential DAF of the wave-in-deck loads, as outlined in Section 9. This procedure reviews the platform static characteristics developed from the static pushover and a few dynamic properties (global and portal natural periods) to assure that the particular platform does not possess some unusual characteristics sensitive to dynamics.

CASE DESCRIPTION	WAVE HEIGHT (ft)		APPLIED LOADS		
	STATIC PUSHOVER	DYNAMIC ANALYSIS	STATIC PUSHOVER FORCE(KIPS)	DYNAMIC ANALYSIS FORCE(KIPS)	RATIO DYN / STAT
Base Cases, Regular Waves :					
3-D No Deck Wave Load :	80.0	78.0	4900	4700	0.96
2-D :					
No Deck Load	80.5	79.0	1390	1350	0.97
With Deck Load	74.5	75.0	1400	1430	1.02
Irregular Seas :					
Synthetic 70-80-70	47.5 *	46.5 *	1390	1380	0.99
Measured N1	47.5 *	69.0 *	1390	1340	0.96
Measured N2	47.5 *	62.0 *	1390	1400	1.01
Measured W1	47.5 *	51.5 *	1390	1390	1.00
Measured W2	47.5 *	45.5 *	1390	1330	0.96
Modified Bracing Configuration :					
Diagonal Braces	73.5	74.5	1250	1350	1.08
Cross Braces	75.0	76.0	1390	1450	1.04
Cross Braces with Mass x 2	73.5	74.5	1310	1360	1.04
Diagonal Braces incl. Portal	72.0	73.5	1230	1300	1.06
Modified Mass and Weight :					
Jacket Mass x 3	73.0	74.5	1330	1400	1.05
Jacket Mass x 3, Damping x 1.5	73.0	75.0	1330	1430	1.08
Jacket Mass x 5	74.0	75.0	1370	1430	1.04
Deck Mass x 2	74.0	77.0	1370	1540	1.12
Deck Mass x 2, Ramped Kinematics	74.0	77.0	1370	1540	1.12
Deck Mass x 3	73.0	76.0	1330	1490	1.12
Deck Mass x 3, No Wave in Deck	80.0	78.0	1380	1330	0.96
Modified Stiffness and Strength :					
Ungrouted Legs	74.0	75.0	1370	1430	1.04
Elastic Modulus x 0.5	57.5	58.0	860	870	1.01
All Leg Thicknesses x 0.75	72.5	75.0	1300	1430	1.10
All Leg Thicknesses x 0.50	70.0	72.0	1180	1280	1.08
Portal Leg Thicknesses x 0.75	72.5	76.0	1300	1460	1.12
Portal Leg Thicknesses x 0.50	69.5	72.0	1160	1280	1.10
Base Case, Damping x 1.5	74.5	75.0	1400	1430	1.02

SUMMARY OF RESULTS

* Crest Elevation, Not W

Table 10-1

Section 11
References

American Petroleum Institute, 1987. "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," Seventeenth Edition.

American Petroleum Institute, 1993. "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design," Twentieth Edition.

Bea, R.G., 1992. "Loading and Capacity Effects in Extreme Storm Waves and Earthquakes," Offshore Technology Conference, Houston, Texas, OTC 7140.

Bea, R.G., and Young, C.N., 1993. "Loading and Capacity Effects on Platform Performance in Extreme Storm Waves and Earthquakes," Proceedings, Offshore Technology Conference, OTC 7140, Houston, Texas.

Det Norske Veritas, Høvik, Norway, Issued 1977. "Rules for the Design, Construction and Inspection of Offshore Structures," Reprinted with Corrections in 1981 and 1982.

PMB Systems Engineering Inc., 1987a. "Development of Inspection and Repair Programs for Fixed Offshore Platforms," Report to Technology Assessment and Research Branch, Minerals Management Service.

PMB Systems Engineering Inc., 1987b. "Development of Platform AIM (Assessment, Inspection, Maintenance) Programs – Phase II," Joint Industry Project.

PMB Systems Engineering, Inc., 1987c. "Hydrodynamic Effects on Design of Offshore Platforms – Phase I," Report to Joint Industry Project.

PMB Systems Engineering Inc, 1988. "Assessment - Inspection - Maintenance of Offsshore Platforms – Phase III," A Joint Industry Project.

PMB Engineering, Inc., 1989. "Hydrodynamic Effects on Design of Offshore Platforms – Phase II, Stage A." Report to Joint Industry Project.

PMB Systems Engineering Inc, 1990. "Assessment - Inspection - Maintenance of Offsshore Platforms – Phase IV," A Joint Industry Project.

PMB Engineering, Inc., 1991. "Hydrodynamic Effects on Design of Offshore Platforms – Phase II, Stage B." Report to Joint Industry Project.

PMB Engineering, Inc, 1993. "Hurricane Andrew – Effects on Offshore Platforms," Joint Industry Project.

Stewart, G., Moan, T., Amdahl, J., Eide, O., Tromans, P., Eberg, E., Hellan, O., Tandberg, T., Hellevig, N., 1993. "Nonlinear Re-assessment of Jacket Structures Under Extreme Cyclic Storm Loading, Parts I - IV," International Offshore Mechanics and Arctic Engineering Symposium.

Appendix A
Literature Search Hardcopy

TIC MISS7 1. doc

DATA COLLECTION : SELECTED ABSTRACTS -1

BRACE AND JOINT BEHAVIOR

Title: Damping in Structural Joints
Author(s): Beards, C. F.
Performing Organization: Imperial Coll. of Science and Technology, London (England).
Sponsoring Organization: National Aeronautics and Space Administration, Washington, DC.
Notes: In Vibration Inst., the Shock and Vibration Digest, Volume 21, No. 4 p 3-5.; Apr 89
Abstract: Friction damping in joints is the major source of inherent damping in most fabricated structures. Although analysis techniques are becoming more refined, it can still be difficult to accurately predict the effect of controlled joint damping on the vibration response of a structure. The range and scope of applications in which it is desirable to provide increased joint damping continues to expand.

Title: Effect of joint flexibility on seismic response parameters of steel jackets.
Author: Einashai, A. S.; Gho, W.
Corporate Source: Imperial Coll, London, Engl
Conference Title: Proceedings of the Second International Offshore and Polar Engineering Conference; San Francisco, CA, USA Conference Date: 1992 Jun 14-19
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 475-480
Abstract: In this paper, the effect of joint flexibility on the collapse mechanism of a typical deep water steel jacket structure is investigated. This is followed by an assessment of the same effect on the dynamic characteristics of the platform, in terms of periods of vibration, mode identification and spacing and modal mass distribution. Finally, the response under earthquake loading is studied, and an estimate of the structural reserve capacity (representative of the ductility and energy absorption capacity) with and without joint flexibility is obtained. It is concluded that joint compliance plays an important part in spectral force calculation, but has no effect on the modal mass distribution. The reserve strength, which is related to the seismic behaviour factor, is over-estimated by using rigid joint analysis results.

Title: RELIABILITY OF TUBULAR JOINTS IN OFFSHORE STRUCTURES.
Author: Packer, Jeffrey A.; Kremer, John S. M.
Corporate Source: Univ of Toronto, Toronto, Ont, Can
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 563-572
Publication Year: 1985
Abstract: The Canadian Standards Association is currently in the process of drafting a Canadian code to govern the design of fixed offshore structures for use in Canadian waters. This paper presents appropriate safety factors for use in such a code, for axially loaded K, T, and Y joints in circular hollow steel sections under predominantly static loading. These safety factors, which rationally incorporate the uncertainties involved with both the loading effect and connection resistance, are developed using a Level II reliability analysis and partial factor optimization. The safety factors (from which connection resistance or performance factors may be derived) are given for the strength equations advocated by four prominent specifications at three different target failure probabilities and for three different loading distributions. (Author abstract) 16 refs.

Title: VIBRATION ANALYSIS OF NAMORADO 2 PLATFORM DURING TOW-OUT.
Author: Rodriguez, Sergio G. Hormazabal; Ebecken, Nelson Favilla
Corporate Source: Petrobras, Braz
Source: Publ by Pentech Press, London, Engl p 594-619 Publication Year: 1986
Abstract: This paper analyses the local vibration of some panels in the Namorado 2 Platform during tow-out operation. According to the inspection report the vibration can be observed, appearing when the barge travels in the same direction as the wind. The influence of the wind on the structure, considering static action and vorticity, on line and perpendicular to the flux, was studied. 6 refs.

BRACE AND JOINT BEHAVIOR

Title: Influence of chord length and boundary conditions on K joint capacity.
Author: Bolt, H. M.; Seyed-Kebari, I. L.; Ward, J. K.
Corporate Source: Billington Osborne-Moss Engineering Ltd, Maidenhead, UK
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 347-354 Publication Year: 1992
Abstract: Recent research has indicated that the ultimate strength and failure characteristics of K joints may vary more significantly with chord length and boundary conditions than had previously been recognised. Many different configurations are contained within the existing experimental isolated joint database and the different influences are difficult to distinguish for the limited dataset. The finite element analyses presented in this paper demonstrate the systematic way in which different factors can be isolated and quantified to ensure that future tests and isolated joint analysis as well as design guidance accurately reflect the constraints on a joint in a frame. (Author abstract)

Title: Nonlinear dynamic response of frame-type structures with hysteretic damping at the joints.
Author: Shi, G.; Atluri, S. N.
Corporate Source: Dalian Univ of Technology, Dalian, China
Source: AIAA Journal v 30 n 1 Jan 1992 p 234-240 Publication Year: 1992
Abstract: The dynamic response of frame-type structures with hysteretic damping at the structural joints, resulting from slipping and nonlinear flexible connections, is investigated in this paper. The slipping at a structural joint is represented by the modified Coulomb joint model. The behavior of a nonlinear flexible connection is modeled by the Ramberg-Osgood function. A simple computational model for the dynamic analysis of frames with the hysteretic damping is presented here. Several numerical examples are included, to illustrate the usefulness of the approach in analyzing large space structures. (Author abstract) 13 Refs.

Title: Analytical and experimental investigations on internally ring stiffened steel tubular joints.
Author: Ramachandra Murthy, D. S.; Madhava Rao, A. G.; Gandhi, P.;
Corporate Source: CSIR Campus, Madras, India
Source: Fatigue Fract Steel Concr Struct ISFF 91 Proc. Publ by Oxford & IBH Pub Co. Pvt. Ltd., New Delhi, India. p 715-728 Publication Year: 1991
Abstract: Offshore structures are susceptible to fatigue failure due to the repetitive action of wave loads. Welded tubular structures commonly used for offshore platforms are subjected to stress concentration at intersection of tubulars, in addition to fatigue loads. Internally ring stiffened tubular joints are found to be efficient in reducing stress concentration and increasing fatigue life. Based on finite element analysis of internally ring stiffened steel tubular T and Y joints, optimum stiffener parameters have been proposed for reducing stress concentration. Parametric formulae have also been developed for calculating maximum stress concentration factors (SCF) for these joints under different loading conditions. Static tests were also conducted on five unstiffened and six internally ring stiffened steel tubular T and Y joints. The tests have proved the efficiency of ring stiffeners in reducing SCF and increasing ultimate strength of the joints. The SCF's predicted by the parametric formulae are found to be in good agreement with the experimental values. The analytical and experimental investigations are discussed in this paper. (Author abstract) 7 Refs.

Title: Elasto-plastic analysis of tubular joints of offshore platforms by finite element method.
Author: Chen, Tieyun; Zhu, Zhenghong
Corporate Source: Shanghai Jiao Tong Univ, Shanghai, China
Source: China Ocean Engineering v 5 n 1 1991 p 13-22 Publication Year: 1991
Abstract: The plastic node method is reformulated by the variational principle and is applied to elasto-plastic finite element analysis of tubular joints, eventually including the effect of internal and external gussets, stiffener rings, etc., if necessary. Four different joints are studied here in detail for the elasto-plastic behavior, the strain at the hot spot, the strain concentration factor around the intersection line, and the propagation of the plastic region with loading up to collapse in order to determine the ultimate

strength, safety factor, and development of the plastic field. The present results are in good agreement with the experimental results. (Author abstract) 10 Refs.

Title: Improved joint model and equations for flexibility of tubular joints.
Author: Ueda, Y.; Rashed, S. M. H.; Nakacho, K.
Corporate Source: Osaka Univ, Osaka, Jpn
Source: Journal of Offshore Mechanics and Arctic Engineering v 112 n 2 May 1990 p 157-168
Publication Year: 1990
Abstract: In tubular frames with simple joints, joints may show considerable flexibility in the elastic as well as the elastic-plastic ranges. Such flexibility may have large effects on the behavior of the structure as a whole. In a previous paper, an effective simple model of tubular joints is developed. The model takes account of joint flexibility in the elastic as well as the elastic-plastic ranges based on elastic-fully plastic load-displacement relatioships. In this paper an improved joint model is presented to provide better accuracy while maintaining simplicity. The accuracy of the model is confirmed through comparisons with results of finite element analysis. Equations to evaluate the initial stiffness of tubular T and Y-joints when braces are subjected to axial compression or in-plane bending moment are also presented. (Edited author abstract) 14 Refs.

Title: Codification of tubular joints design practice. A plea for rationalisation.
Author: Lalani, M.
Corporate Source: Billington Osborne-Moss Engineering Ltd, Ascot, Engl
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium v 3 pt B. Publ by American Soc of Mechanical Engineers (ASME), New York, NY, USA. p 667-675
Publication Year: 1990
Abstract: Large research programmes over the past two decades have generated a significant amount of test data and information on all aspects of static strength, stress analysis and fatigue life evaluation. Whilst the recommendations in various design documents are derived essentially from the same database, a significant number of differences exist in the recommendations, leading to designs which are appreciably different. This paper concentrates on static strength and addresses the background to these differences and their implications to design. Areas of concern are cited and an approach to rationalisation is developed, leading to the specification of future design development and research activities. The use of suitable forums for rationalisation, such as the established Tubular Joints Group and the pending Offshore Eurocode, is described and discussed. (Edited author abstract) 14 Refs.

Title: Ultimate behavior of multiplanar double k-joints of circular hollow section members.
Author: Paul, J. C.; Uena, T.; Makino, Y.; Kurobane, Y.
Corporate Source: Kumamoto Univ, Kumamoto, Jpn
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 377-383 Publication Year: 1992
Abstract: The results of a test on multiplanar double K-joints with circular hollow sections under balanced axial brace loading are reported. The objective of the test is to investigate influences of the brace diameter to chord diameter ratio beta equals d/D, the transverse gap to chord diameter ratio zeta equals g/D, the longitudinal gap to chord thickness ratio zeta equals g/T and the out-of-plane angle phi //1 on the static strength of the joints. These variables are varied in the following range: 0.23 less than equivalent to beta less than equivalent to 0.47, 0.04 less than equivalent to zeta less than equivalent to 0.53, 2 less than equivalent to zeta less than equivalent to 14 and phi //1 equals 60 degree or 90 degree . The influences of zeta and zeta on the ultimate capacity of double K-joints are independent of each other. The ultimate capacity increases with the decrease of zeta but this increase is smaller than for K-joints. The influence of zeta depends on the failure type: for a constant value of beta , the ultimate capacity increases with zeta for the failure type without local deflections of the chord wall between the compression braces and decreases for the failure type with significant local deflections of the chord wall between the compression braces. The ultimate capacity of double K-joints is compared with previous research on double K and T-joints as well as with AWS, IIW and Cidect design recommendations. Formulae for the ultimate capacity of double K-joints are proposed based on past and present results. (Author abstract)

Title: On the assessment of strength of platform with damaged members.
Author: Gu, Yongning; Li, Runpel; Wang, Zhinong
Corporate Source: Shanghai Jiao Tong Univ, Shanghai, China
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 408-414 Publication Year: 1992
Abstract: The axial compressive elastic- plastic behaviour of a tubular beam with combined damages of out-of-straight and local dent has been studied. An incremental-iterative computation procedure is employed to trace the shortening deformation, axial stiffness and stress of a damaged beam at such of axial compressive steps. The effect of initial overall deflection of a beam on the plastic hinge at dent center is discussed. The elastically fixed boundary condition as well as bending moments at beam ends has been introduced into the analysis of ultimate strength of a damaged beam. A method is presented for the strength assessment of a platform with damaged members. (Author abstract) 9 Refs.

Title: Improved formulation for the ultimate static strength of tubular joints subjected to combined loads.
Author: Gu, Hongxin; Chen, Tieyun
Corporate Source: Marine Design and Research Inst of China, Shanghai, China
Source: China Ocean Engineering v 5 n 3 1991 p 255-268 Publication Year: 1991
Abstract: The effect of interaction of loads on the ultimate static strength of tubular joints of offshore fixed platforms, is a practical problem. But there is still absence of rigorous theory to explain available experimental data and empirical criteria for the static strength of tubular joints. The idea of yield at hot spot of tubular joints is introduced in this paper. The interaction equations of plastic capacity for the turbular joints under combined loads (two and three different kinds) are derived. Thereafter the Yura's test data and empirical criteria of ultimate static strength for the tubular joints can be explained. The idea of classification of category of loads in accordance with experimental data and the present theory is suggested. Finally, the improved ultimate capacity equations for tubular joints are recommended. The physical significance of the coefficient of plastic reservation Q//p is discussed. (Author abstract) 8 Refs.

ULTIMATE STRENGTH OF STRUCTURES

Title: Reassessment of Structures: Report 0.1: Cyclic Analyses of 2D Jacket Structure
Author(s): Hellan, O.
Performing Organization: Selskapet for Industriell og Teknisk Forskning, Trondheim (Norway). Div. of Structural Engineering.
Report No: STF71-A90017; ISBN-82-595-5663-4
Abstract: The report describes cyclic analyses of a 2D jacket structure. The objective of the study has been to illustrate a methodology where nonlinear shakedown analyses are used to assess the strength of structures under variable, repeated loading. The methodology is based on FEM formulations for analysis of nonlinear structural response, extended with aspects of shakedown theory and formulations for cyclic plasticity of structural components. For a specified repeated loading, these analyses indicate whether the structure will reach shakedown (responds elastically after a number of elastoplastic cycles), or if the structure is likely to fail due to incremental collapse or reversed plasticity (when the number of cycles approach infinity). The present study shows that cyclic analyses can be used to document elastic structural response (after a number of cycles), even if the structure is loaded beyond static yielding. Under the current design practice, a situation where initial yielding occurs below the design load would require immediate and extensive actions from the operator. Now, cyclic analyses indicate that the structure can be utilized beyond the conventional ULS design limit, and still comply to the regulations' requirements.

Title: Collapse mode of elastic-plastic structures.
Author: Giambanco, F.; Panzeca, T.; Zito, M.
Corporate Source: Univ di Palermo, Palermo, Italy
Source: Journal of Engineering Mechanics v 118 n 6 Jun 1992 p 1083-1092
Abstract: For a structure of elastic-perfectly plastic material subjected to steady and cyclic loads exceeding shakedown limit, the possibility to predict collapse mode, without making a complete analysis, is illustrated. This goal is achieved by using the kinematical part of the solution to the shakedown load factor problem, and by considering that it is proportional to the gradient of the elastic-plastic, steady-state response to cyclic loads at the shakedown limit. A bounding technique, which allows the approximate assessment of any desired measure of plastic deformation occurring in the steady-state phase, is presented. Such technique differs from the usual bounding techniques because the preventive determination of only one bound on a suitable proportionality factor is requested. One the grounds of such bound value, it is possible to compute (with a very small computational effort) other bounds on any chosen measure of plastic deformation, by using the solution of the shakedown load factor problem. (Author abstract) 12 Refs.

Title: Instability and collapse behavior of a seismic structure.
Author: Cheng, Franklin Y.; Ger, Jeng-Fuh
Corporate Source: Univ of Missouri-Rolla, Rolla, MO, USA
Source: Vibration Analysis - Analytical and Computational American Society of Mechanical Engineers, Design Engineering Division (Publication) DE v 37. Publ by ASME, New York, NY, USA. p 215-224
Publication Year: 1991
Abstract: The collapse behavior of a 22-story steel building during the September 19, 1985 Mexico earthquake is investigated by studying hysteretic behavior, ductility factors of individual structural components, and overall instability of the building. The hysteresis models for truss-type girders, bracing members, and box columns to be used in the inelastic analysis of this building are developed. A series of inelastic analyses have been performed for the building by using the multicomponent seismic input of actual Mexico City earthquake records. It was found that the structural response exceeds the original design ductility of this building because most girders in the building have suffered large ductilities. Due to the load redistribution effects from the ductile-failed girders, local bucklings developed at many columns on floors 2, 3, and 4. Therefore, most columns on floors 2 through 4 lost their load carrying capacities and rigidities which then caused the building to tilt and rotate. As a result, more columns on floors 5 through 7 developed local buckling and more bracing members buckled. It is believed that ductile failures of girders combined with the local bucklings of columns in the lower part of the building resulted

in significant story drift, building tilt, P- Delta effect, and the failure mechanism. (Author abstract) 12 Refs.

Title: METHODOLOGIES FOR ULTIMATE LIMIT STATE RELIABILITY ANALYSIS OF OFFSHORE JACKET PLATFORMS.
Author: Edwards, G.; Heidweiller, A.; Kerstens, J.; Vrouwenvelder, A.
Corporate Source: Shell Research, Rijswijk, Neth
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 564-568 Publication Year: 1985
Abstract: A typical plane frame from a fixed offshore platform has been used as a case study and its ultimate limit state system reliability under extreme wave loading considered. Both classical and Bayesian inference have been applied to estimate the failure probability from Monte Carlo results; Level II procedures have been used to analyse the derived failure mode expressions. Furthermore Importance Sampling has been considered. The results suggest that a dual approach, where Monte Carlo is combined with Level II analysis on dominant failure modes, may offer advantages in system reliability analysis in the future. (Author abstract) 9 refs.

Title: Static and dynamic analysis of collapse behaviour of steel structures.
Author: Wada, Akira; Kubota, Hideyuki
Corporate Source: Tokyo Inst of Technology, Yokohama, Jpn
Source: Computer Methods in Applied Mechanics and Engineering v 91 n 1-3 Oct 1991. p 1365-1378 Publication Year: 1991
Abstract: This paper presents the computer program to pursue nonlinear behavior of steel structures until collapse state statically and dynamically and it shows two results of example analysis. The program is specially designed to use the vector-processing function of supercomputer quite effectively. ETA10 and CRAY-2 supercomputers areused in the examples. 7 Refs.

Title: EARTHQUAKE RESPONSE OF STEEL PLATFORM MODELS WITH LOCAL NONLINEARITY.
Author: Liu, Yin-bo; Chen, Dan
Corporate Source: Tsinghua Univ, Tsinghua, China
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 415-421 Publication Year: 1988
Abstract: The paper presents two kinds of local nonlinear concepts for piled jacket platform for resisting intensive earthquakes. Shaking table test and theoretical analysis are carried out on two steel platform models which are designed on one of those two local nonlinear concepts respectively. Also, responses of the two models under extreme earthquakes are calculated by computer to analyze the anti-collapse capacity. The results show that the local nonlinear properties are effective in controlling the earthquake responses. And the structure which has local nonlinear properties reveals good behavior on resisting collapse. (Author abstract) 5 refs.

Title: INTEGRATED DYNAMIC ANALYSIS APPROACH FOR OFFSHORE FATIGUE DAMAGE PREDICTION AND STRUCTURAL MEMBER DESIGN.
Author: Alves, J. L. D.; Torres, A. L. F. L.; Lima, E. C. P.; Ebecken, N. F.
Corporate Source: COPPE, Braz
Source: Publ by Springer-Verlag, Berlin, West Ger and New York, NY, USA p 497-505 Publication Year: 1986
Abstract: The use of a full dynamic analysis in the frequency domain, applying the modal superposition method with a pseudo-static correction technique is applied for structural design of offshore platforms. This method provides accurate results without great computational effort, accounting the dynamic behaviour and the spatial structure characteristics. It should be observed that member failure and joint capacity (punching shear) checking are related to the total stress combination (static PLUS dynamic stresses). 5 refs.

Title: CATCHMENT REGIONS OF MULTIPLE DYNAMIC RESPONSES IN NONLINEAR PROBLEMS OF OFFSHORE MECHANICS.
Author: Virgin, L. N.; Bishop, S. R.
Corporate Source: Univ of London, London, Engl
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 15-22 Publication Year: 1988
Abstract: Consideration of nonlinear effects in the dynamic analysis of offshore structures leads to complex phenomena even for relatively simple deterministic mathematical models. Nonlinearity in the stiffness or restoring force of a structure can be conveniently incorporated into a differential equation of motion which can then be solved either analytically, using perturbation techniques, or numerically for various parameter ranges and forcing conditions. A particular feature of nonlinear dynamics is the appearance of multiple, competing steady-state oscillations which depend crucially on initial conditions. This paper concentrates on the computation of the domains of attraction or catchment regions using numerical techniques based on Poincare mapping ideas. The three specific examples illustrated are the roll motion of a ship, the oscillations of an articulated column and the surge response of a moored semi-submersible. (Edited author abstract) 17 refs.

Title: Optimal nonlinear dynamic analysis of steel jacket structures.
Author: Izzuddin, B. A.; Elnashai, A. S.
Corporate Source: Imperial Coll, London, Engl
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 272-277 Publication Year: 1992
Abstract: A new and efficient method for the nonlinear dynamic analysis of steel structures is presented in this paper. The method employs a newly developed quartic formulation which is capable of representing the large displacement behaviour of elastic beam-columns using one element per member. The efficiency of the procedure is further enhanced in the elasto-plastic range through the use of automatic mesh refinement, where the analysis is always started using a coarse mesh to be refined during analysis when and where necessary. Additionally, automatic adjustment of the integration time-step is employed, since such a process is shown to contribute significantly to the efficiency of the nonlinear dynamic analysis of jacket structures. Verification examples using ADAPTIC, a general purpose program for the nonlinear analysis of framed structures subjected to earthquake, blast and impact loads, demonstrate the accuracy and significant computational savings of the proposed analysis method. (Author abstract) 10 Refs.

Title: Nonlinear dynamic response of framed structures using the mode superposition method.
Author: Fahmy, Mohamed W.; Namini, Ahmad H.
Corporate Source: Univ of Miami, Coral Gables, FL, USA
Source: Proceedings of Engineering Mechanics. Publ by ASCE, New York, NY, USA. p 457-460 Publication Year: 1992
Abstract: This paper describes a methodology for nonlinear dynamic response analysis of framed structures using the mode superposition method. A portal frame example is presented and compared to classical methods for determination of critical buckling loads. Results indicate that the critical buckling load via a dynamic analysis is one-half that of a static analysis due to a dynamic magnification factor of two for long duration dynamic loads of constant magnitude. (Author abstract) 11 Refs.

Title: 3D frequency domain analysis of offshore structures.
Author: McNamara, J. F.; Lane, M.
Corporate Source: Univ Coll Galway, Irel
Source: Proceedings of Engineering Mechanics. Publ by ASCE, New York, NY, USA. p 192-195 Publication Year: 1992
Abstract: This paper reports the application of three-dimensional (3D) frequency domain computational tools to the dynamic analysis of offshore structure systems. Previous applications of 3D frequency analysis by the present authors in the areas of TLP tether and jack-up analysis are summarised. Results are presented here for a new area of application, namely a deep-water multi-line steep-S flexible riser

configuration. The results are compared with time domain calculations and, in general, excellent agreement is reported. A nonlinear static option, which permits linear dynamic motions about a nonlinear static position, is outlined as an efficient extension of the basic frequency domain method. The present work addresses an area where frequency domain analysis has not previously been systematically exploited, and indicates excellent potential for future innovative applications. (Author abstract) 4 Refs.

Title: RELIABILITY OF TUBULAR JOINTS IN OFFSHORE STRUCTURES.
Author: Packer, Jeffrey A.; Kremer, John S. M.
Corporate Source: Univ of Toronto, Toronto, Ont, Can
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 563-572
Publication Year: 1985
Abstract: The Canadian Standards Association is currently in the process of drafting a Canadian code to govern the design of fixed offshore structures for use in Canadian waters. This paper presents appropriate safety factors for use in such a code, for axially loaded K, T, and Y joints in circular hollow steel sections under predominantly static loading. These safety factors, which rationally incorporate the uncertainties involved with both the loading effect and connection resistance, are developed using a Level II reliability analysis and partial factor optimization. The safety factors (from which connection resistance or performance factors may be derived) are given for the strength equations advocated by four prominent specifications at three different target failure probabilities and for three different loading distributions. (Author abstract) 16 refs.

Title: RELIABILITY-BASED ANTISEISMIC ANALYSIS OF STRUCTURES.
Author: Kobori, Takuji
Corporate Source: Kyoto Univ, Kyoto, Jpn
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 1-22
Publication Year: 1985
Abstract: Some basic problems associated with the estimation of reliable safety of a critical structure such as the reactor building of a nuclear power plant are investigated from the viewpoint of engineering seismology and earthquake-resistant design. First, we present the method to provide synthetic earthquake ground motion model with consideration of uncertain earth structure and source mechanism to be used throughout the seismic response analysis of this paper. Second, the analytical technique is developed to be available for nonlinear seismic response of soil-structure system on the basis of Ito's stochastic differential equation. The reliability of the structure is estimated by the probability density function of absolute maximum response and first passage failure. Finally, the total seismic safety of a critical structure is examined by the response of the interaction system composed of the upper main structure, the foundation system, the boundary layer and the soil medium to the earthquake ground motion model. (Author abstract) 13 refs.

Title: SYSTEM RELIABILITY ANALYSIS OF A JACKET STRUCTURE.
Author: Baadshaug, Ola; Bach-Gansmo, Ove
Corporate Source: AS Veritec, Hovik, Norw
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 613-617
Publication Year: 1985
Abstract: An approach for structural system reliability analysis is described. This is a failure mode analysis, in which the structural elements are successively 'unzipped'. Some critical aspects regarding system analysis of offshore structures are treated. The approach has been used in an analysis of a jacket structure, and the main conclusions from this study are reported. (Author abstract) 10 refs.

Title: SEISMIC RELIABILITY ANALYSIS OF HYSTERETIC STRUCTURES BASED ON STOCHASTIC DIFFERENTIAL EQUATIONS.
Author: Suzuki, Yoshiyuki; Minai, Ryoichiro
Corporate Source: Kyoto Univ, Uji, Jpn
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 177-186
Publication Year: 1985

Abstract: An analytical method for determining the stochastic seismic response, including the safety measures, and the reliability functions of hysteretic structures is presented. The safety measures such as the maximum ductility ratio, the cumulative plastic deformation, cumulative hysteretic energy and low-cycle fatigue damage are described by the nonlinear differential forms as well as the hysteretic characteristics. Then, the state equations of overall dynamic system can be expressed by the Ito stochastic differential equations. By introducing the approximate probability density, function, the moment equations are solved, and the statistics of the safety measures and the reliability functions are evaluated. The analytical method is illustrated for the bilinear hysteretic structures to nonstationary excitations. (Author abstract) 9 refs.

Title: METHODOLOGIES FOR ULTIMATE LIMIT STATE RELIABILITY ANALYSIS OF OFFSHORE JACKET PLATFORMS.
Author: Edwards, G.; Heidweiller, A.; Kerstens, J.; Vrouwenvelder, A.
Corporate Source: Shell Research, Rijswijk, Neth
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 564-568
Publication Year: 1985
Abstract: A typical plane frame from a fixed offshore platform has been used as a case study and its ultimate limit state system reliability under extreme wave loading considered. Both classical and Bayesian inference have been applied to estimate the failure probability from Monte Carlo results; Level II procedures have been used to analyse the derived failure mode expressions. Furthermore Importance Sampling has been considered. The results suggest that a dual approach, where Monte Carlo is combined with Level II analysis on dominant failure modes, may offer advantages in system reliability analysis in the future. (Author abstract) 9 refs.

Title: Incremental collapse of structures with constant plus cyclically varying loads.
Author: Guralnick, Sidney A.; Erber, Thomas; Soudan, Osama; He, Jixing
Corporate Source: Illinois Inst of Tech., Chicago, IL, USA
Source: Journal of Structural Engineering v 117 n 6 Jun 1991 p 1815-1833
Publication Year: 1991
Abstract: Energy methods previously developed for the shakedown analyses of framed structures are extended to include structures subjected to cyclically varying loads in the presence of constant, or bias, loads. This approach is based on the hypothesis that, if the total hysteresis energy absorbed by a structure during an indefinitely prolonged repetitive loading program is unbounded, then the structure must ultimately fail. This hypothesis leads to results that are entirely consistent with the classical shakedown theorems. Several illustrative examples demonstrate that the incremental collapse envelopes of framed structures subjected to cyclically varying patterns of loading can be drastically reduced when constant loads are present. These results are essentially due to inelastic interactions between the cyclic and constant loads that magnify the effects of hysteresis. In fact, the reduction in the safe loading ranges (as defined by the incremental collapse envelope) can be a sensitive function of the constant, or bias, load components. (Author abstract) 28 Refs.

Title: Determination of the collapse load of plastic structures by the use of an upper bounding algorithm.
Author: Avdelas, A. V.
Corporate Source: Aristotle Univ, Thessaloniki, Greece
Source: Computers and Structures v 40 n 4 1991 p 1003-1008
Abstract: An upper bounding algorithm is applied to the problem of the elastoplastic analysis of structures expressed as linear complementarity problems. By the use of this algorithm, the time consuming procedure of solving large quadratic optimization problems can be avoided. Applications close the paper. (Author abstract) 25 Refs.

DATA COLLECTION : SELECTED ABSTRACTS -2

ULTIMATE STRENGTH OF STRUCTURES:

Title: Nonlinear analyses of three-dimensional steel frames with semi-rigid connections.
Author: Hsieh, S. -H.; Deierlein, G. G.
Corporate Source: Cornell Univ, Ithaca, NY, USA
Source: Computers and Structures v 41 n 5 1991 p 995-1009 Publication Year: 1991
Abstract: With the ultimate aim of improved methods for the realistic limit state design of structures, a method has been developed for incorporating nonlinear connection response in the analysis of three-dimensional steel structures. The method is implemented in an interactive graphics analysis and design program which can model both geometric and material nonlinearities in framed structures. The connection model includes nonlinear moment-rotation response for both major- and minor-axis bending. Standardized models for different types of connections which are calibrated to existing test data and are amenable to design are presented. Application of the method is presented for a realistic case study consisting of a low-rise three-dimensional structure with partially restrained connections. (Author abstract) 19 Refs.

Title: Random vibration and first passage failure.
Author: Roberts, J. R.
Corporate Source: Univ of Sussex, Sussex, Engl
Source: Courses and Lectures - International Centre for Mechanical Sciences n 317 1991 p 1-49 Publication Year: 1991
Abstract: In the first part of this Chapter, a variety of techniques for predicting nonlinear dynamic system response to random excitation are discussed. These include methods based on modelling the response as a continuous Markov process, leading to diffusion equations, statistical linearization, the method of equivalent nonlinear equations and closure methods. Special attention is paid to the stochastic averaging method, which is a combination of an averaging technique and Markov process modelling. It is shown that the stochastic averaging method is particularly useful for estimating the 'first-passage' probability that the system response stays within a safe domain, within a specified period of time. Results obtained by this method are presented for oscillators with both linear and nonlinear damping and restoring terms. An alternative technique for solving the first-passage problem, based on the computation of level crossing statistics, is also described: this is especially useful in more general situations, where the stochastic averaging is inapplicable. (Author abstract) 46 Refs.

Title: Optimal nonlinear dynamic analysis of steel jacket structures.
Author: Izzuddin, B. A.; Elnashai, A. S.
Corporate Source: Imperial Coll, London, Engl
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 272-277 Publication Year: 1992
Abstract: A new and efficient method for the nonlinear dynamic analysis of steel structures is presented in this paper. The method employs a newly developed quartic formulation which is capable of representing the large displacement behaviour of elastic beam-columns using one element per member. The efficiency of the procedure is further enhanced in the elasto-plastic range through the use of automatic mesh refinement, where the analysis is always started using a coarse mesh to be refined during analysis when and where necessary. Additionally, automatic adjustment of the integration time-step is employed, since such a process is shown to contribute significantly to the efficiency of the nonlinear dynamic analysis of jacket structures. Verification examples using ADAPTIC, a general purpose program for the nonlinear analysis of framed structures subjected to earthquake, blast and impact loads, demonstrate the accuracy and significant computational savings of the proposed analysis method. (Author abstract) 10 Refs.

Title: Ultimate limit states with combined load processes.
Author: Ritner-Gregersen, Elzbieln M.; Haver, Sverre; Loseth, Robert
Corporate Source: AS Veritas Research, Hovik, Norw
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 515-522 Publication Year: 1992
Abstract: The overall objective of the design of a marine structure is to ensure that the structure can resist all foreseen loads with an adequate degree of safety against failure. In this connection the most important load processes, wind, waves and current, are often assumed to be fully (or nearly fully) correlated. The assumption of full correlation is most probably rather conservative and may result in an overestimation of the design loads. This study investigates the effects of utilizing a joint environmental model of wind, waves and current when predicting the extreme response of a deep water jacket and compares the results with the results obtained using design practice. The joint environmental model is limited to the following environmental parameters: 1-hour mean wind speed, current speed, significant wave height (sea and swell), spectral peak period (sea and swell), main wave direction (wind and current are assumed to be collinear with the main wave direction), and is fitted to instrumental data (1980-1985) and hindcast data (1955-1985) from Haltenbanken (65 degree 0 prime N, 7 degree 36 prime E). The extreme values of the axial stress in a selected structural member are considered. The effects of using different models for the wave spectrum are indicated and compared to the effects of utilizing a simultaneous description of the environmental parameters. (Author abstract) 10 Refs.

Title: Equivalent linearisation and fatigue reliability estimation for offshore structures.
Author: Melchers, R. E.; Ahammed, M.
Corporate Source: Univ of Newcastle, Newcastle, Aust
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 531-536 Publication Year: 1992
Abstract: The force effects due to wave action on offshore structures are conventionally determined using Morison's equation. This is a nonlinear relation in velocity, conventionally represented by an equivalent linearisation for converting wave velocities to forces, necessary to allow structural dynamic analysis by frequency domain methods (the most commonly applied approach). The suggested approaches for the linearisation can be shown to be of essentially two classes; those equivalent to Borgman's approach and those equivalent to Bolotin's. In general, it is not clear, a priori, which is more the appropriate. Nor is it clear whether these classical linearisations are relevant, particularly when structural fatigue is to be estimated (e.g., through the Palmgren-Miner rule). A criterion for derivation of a linearisation constant under fatigue considerations can be derived relatively simply. Unfortunately the criterion is not easily applied except by iteration. A sensitivity study has been performed for a simple but realistic offshore structure to estimate the relative importance of correctly estimating the linearisation constant. (Author abstract)

Title: Computational strategies for the nonlinear dynamic analysis of compliant deepwater structures.
Author: Jacob, Brena Pinheiro; Favilla Ebecken,; Nelson Francisco
Corporate Source: COPPE-Federal Univ of Rio de Janeiro, Rio de Janeiro, Braz
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 27-34 Publication Year: 1992
Abstract: This work presents the application of recent developments on numerical strategies to the nonlinear dynamic analysis of compliant structures for deepwater oil exploration and production. These developments comprise an adaptive direct time integration procedure, which automatically adjusts time step values and triggers stiffness reevaluations, and an adaptive reduced integration method applied to a nonlinear incremental-iterative formulation. The positive characteristics of the proposed numerical tools are evaluated based on the results of the presented applications, which include the analysis of a flexible riser configuration, and of the full-scale model of a deepwater compliant guyed tower. (Author abstract)

EARTHQUAKE ANALYSIS STATE OF PRACTICE

Title: Structural model correlation using large admissible perturbations in cognate space.
Author: Bernitsas, Michael M.; Tawekal, Ricky L.
Corporate Source: Univ of Michigan, Ann Arbor, MI, USA
Source: AIAA Journal v 29 n 12 Dec 1991 p 2222-2232
Abstract: A nonlinear perturbation method is developed to solve the problem of correlating a finite element model (FEM) to a struture for which an incomplete set of natural frequencies and mode shapes and/or some static deflections have been measured. The solution algorithm can handle differences between FEM and struture, in design variables and response, as large as 100-300%, depending on the scale of the structure and correlation measures. This is achieved incrementally by making inadmissible predictions, identifying the modal cognate space relevant to the correlation measures, and making admissible corrections in the cognate space. The developed computer code postprocesses results of the FEM modal and/or static analyses of the initial model only. No additional finite element analysis is required. Lagrange multipliers reveal the dominant correlation requirements and the active admissible cognate subspace. Depending on the number of correlation variables and measures, an optimal, a unique, or an inadmissible minimal error solution may be produced. Beam and offshore tower examples are used to test the algorithm and investigate conflicting requirements, definition of admissible cognate space, limits of allowable differences between FEM and structure, accuracy, and cost of the nonlinear perturbation method. (Author abstract) 29 Refs.

Title: Effect of earthquakes on structures, utilities and engineering services.
Author: Hutchinson, G. L.; Irvine, H. M.
Corporate Source: Univ of Melbourne, Aust
Source: National Conference Publication - Institution of Engineers, Australia v 1 n 91 pt 1. Publ by IE Aust, Barton, Aust. p 35-53 Publication Year: 1991
Abstract: The total strain energy released during an earthquake is known at the magnitude of the earthquake and it is measured on the Richter scale. It is defined quite simply as the amplitude of the recorded vibrations on a particular kind of seismometer located at a particular distance from the epicentre. The amount of earthquake energy released is logarithmically related to the Richter number such that each unit increase in the Richter number means that the energy released increases 32 times and the amplitude of ground motion is increased ten-fold. The magnitude of an earthquake reflects the size of energy released at the source of the earthquake, but it does not indicate whether structural damage can be expected at a particular site. The local intensity of a particular earthquake is measured on the subjective Modified Mercalli scale which ranges from 1 (barely felt) to 12 (total destruction). The Modified Mercalli scale is essentially a means by which damage may be assessed after an earthquake. In a given location, where there has been some experience of the damaging effects of earthquakes, albeit only subjective and qualitative, regions of varying seismic risk may be identified. 10 Refs.

Title: CURRENT EARTHQUAKE ENGINEERING RESEARCH IN THE CIVIL ENGINEERING DEPARTMENT, UNIVERSITY OF AUCKLAND.
Author: Pender, M. J.
Corporate Source: Univ of Canterbury, NZ
Source: v 1. Publ by Inst of Professional Engineers New Zealand Inc, Wellington, NZ p 66-69 Publication Year: 1987
Abstract: This paper deals with current earthquake engineering research at the University of Auckland, Australia. Both geotechnical and structural engineering aspects are covered. Some of the topics discussed are earthquake soil-structure interaction; cyclic stress-strain behavior of clay soils; seismic response of pile foundations; response of reinforced concrete structures; and behavior of timber brick veneer walls under cyclic loading. Other topics covered are prediction of seismic response, finite element modeling of beams, and energy dissipators of cross-braced frames.

Title: DETERMINATION OF MODAL DAMPING RATIOS AND NATURAL FREQUENCIES FROM BISPECTRUM MODELING.
Author: Liou, Cheng-Yuan; Chang, Chih-Chi
Corporate Source: Natl Taiwan Univ, Taipei, Taiwan
Source: Oceans (New York) 1987. Publ by IEEE, New York, NY, USA. Available from IEEE Service Cent (Cat n 87CH2498-4), Piscataway, NJ, USA p 548-553 Publication Year: 1987
Abstract: The authors present a signal processing and testing technique to determine the damping ratio and natural frequency of offshore structures excited by both unexpected Gaussian forces and known non-Gaussian driving forces. Due to the unexpected exciting forces, such as turbulence in the ocean environment, the transfer functions of the offshore structures can not be hardly determined through operating a known driving force and measuring its response. In order to overcome this problem, some previous techniques model the unexpected forces as white-Gaussian forces or almost-white-Gaussian forces and determine the modal parameters from the response only, and others average the input and output to effectively suppress unexpected parts. The author's method uses third-order moment to eliminate the influence of the unexpected Gaussian forces from the determination of the transfer function of a structure which has linear properties. The third-order moment property of the response function is modeled by a bispectral model. The modal parameters can be calculated from the estimated model's coefficients. 9 refs.

Title: REPRESENTATION OF NON-STATIONARY GROUND MOTION ACCELERATION USING A TIME-VARYING MODEL.
Author: Sharma, Moham P.; Shah, Haresh C.
Corporate Source: Stanford Univ, Stanford, CA, USA
Source: Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 521-525 Publication Year: 1985
Abstract: A non-stationary representation of ground motion acceleration is formulated through the use of time dependent auto-regressive models. The time dependence of the model is achieved by modulating white noise by a time varying gain which is then used as input to a time varying AR filter. An iterative scheme is described which identifies both the time varying gain and the time varying AR coefficients, given the acceleration record. In addition to being useful in representing the given acceleration record and the time dependent AR coefficients the gain can be used to generate synthetics with time dependent spectra which are similar to that of the original record. (Author abstract) 6 refs.

Title: Seismic-energy dissipation in MDOF structures.
Author: Leger, Merre; Dussault, Serge
Corporate Source: McGill Univ, Montreal, Que, Can
Source: Journal of Structural Engineering v 118 n 5 May 1992 p 1251-1269 Publication Year: 1992
Abstract: The seismic input energy imparted to a structure is dissipated by hysteretic behavior and other nonyielding mechanisms usually represented by equivalent viscous damping. It is generally recognized that there is a strong correlation between the energy dissipated by hysteretic action and the seismically induced level of damage. While viscous damping has a small effect on the amount of energy imparted to a structure, it has a significant influence on the amount of hysteretic energy dissipation. A parametric study is presented on the influence of the mathematical modeling of viscous damping on seismic-energy dissipation of multidegree-of-freedom (MDOF) structures. The damping is modeled using massproportional, stiffness-proportional, and Rayleigh damping computed from either the initial elastic or the tangent inelastic system properties. Various structural performance indices are evaluated for bilinear hysteresis model of simple MDOF structures with different strength levels, strain hardening ratios, and damping ratios. (Author abstract) 29 Refs.

Title: Jackup structures nonlinear forces and dynamic response.
Author: Winterstein, Steven R.; Loseth, Robert
Corporate Source: Stanford Univ, Stanford, CA, USA
Conference Title: Proceedings of the 3rd IFIP WG 7.5 Conference on Reliability and Optimization of Structural Systems '90

Conference Location: Berkeley, CA, USA Conference Date: 1990 Mar 26-28
Source: Lecture Notes in Engineering n 61. Publ by Springer-Verlag Berlin, Dept ZSW, Berlin 33, Ger. p 350-358

Abstract: Simple analytical methods are shown for stochastic nonlinear dynamic analysis of offshore jacket and jackup structures. Base shear forces are first modelled, and then imposed on a linear 1DOF structural model to predict responses such as deck sway. The force model retains the effects of nonlinear wave kinematics and Morison drag on base shear moments, extremes, and spectral densities. Analytical models are also given for response moments and extremes. Good agreement with simulation is found for a sample North Sea jackup. The effects of variations in environmental and structural properties are also studied. (Author abstract) 12 Refs.

Title: Nonlinear dynamic analysis of damped structures using the transfer matrix technique.
Author: Akintilo, I. A.
Corporate Source: Iaa Associates, London, Engl
Source: Engineering Structures v 14 n 3 1992 p 180-187 Publication Year: 1992

Abstract: A conceptually simple and computationally efficient numerical model, based on the transfer matrix formulation is proposed for the prediction of nonlinear dynamic responses of a coupled shear wall structure incorporating damping. With the transfer matrix method, damping is introduced into the formulation through the field and station transfer matrices using the constitutive law. The effect of damping on the responses of a vibrating beam is incorporated into the station transfer matrix while those of the wall are incorporated into the field transfer matrix. The nonlinear behaviour is approximated as a sequence of successively changing linear systems over a short time interval. The transfer matrix formulation adopts established models of material behaviour in dealing with the inelastic beam element deformation and load reversals. The dynamic solution assumes constant properties within short time increments, and relies on the kinematic relationships of the Wilson theta method. (Author abstract) 32 Refs.

Title: Stress-response of offshore structures by equivalent polynomial expansion techniques.
Author: Sigurdsson, G.; Nielsen, S. R. K.
Corporate Source: Univ of Aalborg, Aalborg, Den
Source: Proc First 90 Eur Offshore Mech Symp. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 134-140 Publication Year: 1990

Abstract: This paper concerns an investigation of the effects of nonlinearity of drag loading on offshore structures excited by 2D wave fields, where the nonlinear term in the Morison equation is replaced by an equivalent cubic expansion. The equivalent cubic expansion coefficients for the equivalent drag model are obtained using the least mean square procedure. Numerical results are given. The displacement response and the stress response processes obtained using the above loading model are compared with simulation results and those obtained from equivalent linearization of the drag term. (Author abstract) 8 Refs.

Title: EARTHQUAKE RESPONSE OF STEEL PLATFORM MODELS WITH LOCAL NONLINEARITY.
Author: Liu, Yin-bo; Chen, Dan
Corporate Source: Tsinghua Univ, Tsinghua, China
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 415-421 Publication Year: 1988

Abstract: The paper presents two kinds of local nonlinear concepts for piled jacket platform for resisting intensive earthquakes. Shaking table test and theoretical analysis are carried out on two steel platform models which are designed on one of those two local nonlinear concepts respectively. Also, responses of the two models under extreme earthquakes are calculated by computer to analyze the anti-collapse capacity. The results show that the local nonlinear properties are effective in controlling the earthquake responses. And the structure which has local nonlinear properties reveals good behavior on resisting collapse. (Author abstract) 5 refs.

Title: STRUCTURE-WAVE INTERACTION UNDER EARTHQUAKE EXCITATION.
Author: Kokkinowrachos, K.; Thanos, I
Corporate Source: RWTH, Aachen, West Ger
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 405-414 Publication Year: 1988
Abstract: A method for the hydrodynamic analysis of large bottom-fixed offshore structures under earthquake action is presented. The investigation deals with arbitrarily shaped vertical bodies of revolution, to which the so-called macroelement method can be applied. The structure is considered here as rigid, the compressibility of the surrounding water has been taken into account. Numerical results for several bottom-mounted structures give information on the earthquake induced hydrodynamic forces and their parts (added mass and damping). The effectiveness of the macroelement method is demonstrated. The influence of geometrical shape, relative dimensions, water depth and water compressibility is shown. (Author abstract) 23 refs.

Title: NEW RATIONAL ENGINEERING APPROACHES TO THE EARTHQUAKE EVENT AND ITS RELATION TO THE DESIGN OF OFFSHORE STRUCTURES.
Author: Borg, S. F.
Corporate Source: Stevens Inst of Technology, Hoboken, NJ, USA
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 553-562 Publication Year: 1985
Abstract: A new approximate earthquake engineering theory is applied to the structural analysis and design problem for offshore platform structures (OPS). The theory is founded upon the fundamental premise that 'energy' is the key parameter in the earthquake event - from its initiation, the 'mechanism', up until its final effect upon structures. New design charts are described and the detailed design procedure is outlined for two of the several different types of support structures for OPS. (Author abstract) 8 refs.

Title: RELIABILITY-BASED ANTISEISMIC ANALYSIS OF STRUCTURES.
Author: Kobori, Takuji
Corporate Source: Kyoto Univ, Kyoto, Jpn
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 1-22 Publication Year: 1985
Abstract: Some basic problems associated with the estimation of reliable safety of a critical structure such as the reactor building of a nuclear power plant are investigated from the viewpoint of engineering seismology and earthquake-resistant design. First, we present the method to provide synthetic earthquake ground motion model with consideration of uncertain earth structure and source mechanism to be used throughout the seismic response analysis of this paper. Second, the analytical technique is developed to be available for nonlinear seismic response of soil-structure system on the basis of Ito's stochastic differential equation. The reliability of the structure is estimated by the probability density function of absolute maximum response and first passage failure. Finally, the total seismic safety of a critical structure is examined by the response of the interaction system composed of the upper main structure, the foundation system, the boundary layer and the soil medium to the earthquake ground motion model. (Author abstract) 13 refs.

Title: Optimal nonlinear dynamic analysis of steel jacket structures.
Author: Izzuddin, B. A.; Elnashai, A. S.
Corporate Source: Imperial Coll, London, Engl
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 272-277 Publication Year: 1992
Abstract: A new and efficient method for the nonlinear dynamic analysis of steel structures is presented in this paper. The method employs a newly developed quartic formulation which is capable of representing the large displacement behaviour of elastic beam-columns using one element per member. The efficiency of the procedure is further enhanced in the elasto-plastic range through the use of automatic mesh refinement, where the analysis is always started using a coarse mesh to be refined during analysis when and where necessary. Additionally, automatic adjustment of the integration time-step is

employed, since such a process is shown to contribute significantly to the efficiency of the nonlinear dynamic analysis of jacket structures. Verification examples using ADAPTIC, a general purpose program for the nonlinear analysis of framed structures subjected to earthquake, blast and impact loads, demonstrate the accuracy and significant computational savings of the proposed analysis method. (Author abstract) 10 Refs.

Title: VIBRATION TESTING AND DYNAMIC ANALYSIS OF OFFSHORE PLATFORM MODELS.
Author: Zhao, Yin; Chen, Zhongyi; Ke, Renqun
Corporate Source: Nanjing Hydraulic Research Inst, Nanjing, China
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 85-89 Publication Year: 1988
Abstract: This paper describes the application of an experimental modal analysis technique to the models of a deepwater jacket fixed platform and a jack-up drilling platform with a foundation mat and presents the results of testing and analysis. Comparisons have been made with finite element analyses and other dynamic analyses. In addition, dynamic displacement responses of the model platforms have been measured when subjected to waves in a wave basin. (Author abstract) 6 refs.

Title: IMPROVED REDUCED BASIS TECHNIQUE FOR DYNAMIC ANALYSIS OF OFFSHORE STRUCTURES.
Author: Coutinho, A. L. G. A.; Landau, L.; Alves, J. L. D.; Lima, E. C. P.;
Corporate Source: Federal Univ of Rio de Janeiro, Braz
Source: Publ by Pentech Press, London, Engl p 514-530 Publication Year: 1986
Abstract: The reduced basis technique for dynamic analysis of structures through a suitable coordinate transformation is one of the most attractive alternative for large structural systems. Earlier approaches employ the modal matrix as the coordinate transformation matrix. These approaches are known as modal superposition methods. In this work, a coordinate transformation based on a set of orthonormal vectors generated by a Lanczos-type algorithm is examined. It is shown, in two typical offshore applications, that the proposed approach reduces remarkably the computational effort spent on dynamic response computations without loss of accuracy. Furthermore, by an adequate choice of the Lanczos algorithm starting vector, it is also seen that the localized effects, which depend on the higher modes, are well represented. (Author abstract) 12 refs.

Title: Unified dynamic substructure method with Lanczos vectors for offshore structures.
Author: Zheng, Zhao-chang; Lu, Jia; Xie, Geng
Corporate Source: Tsinghua Univ, Beijing, China
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 453-457 Publication Year: 1992
Abstract: A unified dynamic substructure method for dynamic analysis of offshore jacket structures is reported herein. Using the constrained modes and normal modes or Lanczos vectors, a new modal transformation developed from hybrid method is given, in which the interface forces are replaced by interface displacements. The first synthesis equation is assembled easily as fixed interface method, yet the interface coordinate can be further eliminated as the free-interface method. The new method unifies the fixed- and free- interface methods as well as hybrid method, permits easy implementation of Lanczos vectors to replace normal modes so that only static modes are calculated. The dynamic substructure method thus becomes more flexible and efficient. This method has been used in the dynamic analysis of offshore jacket structures. (Author abstract)

Title: Modal analysis of vibration response for condition monitoring of structures.
Author: Hearn, George
Corporate Source: Univ of Colorado at Boulder, Boulder, CO, USA
Source: Proceedings of Engineering Mechanics. Publ by ASCE, New York, NY, USA. p 940-943 Publication Year: 1992

Abstract: The response of a structure to load is evidence of what the structure is; of the materials, members, connections and form of the structure. Response is evidence as well of the condition of a structure. As a structure deteriorates its response to load will change, and this change can be used to monitor structural conditions. Condition monitoring of structures by modal analysis has been applied with success in three laboratory studies. A central feature of the method is the formation of a database of modal parameter changes for expected deterioration events. 7 Refs.

Title: **PROBABILISTIC ANALYSIS OF HYSTERETIC STRUCTURAL EARTHQUAKE RESPONSE.**
Author: Asano, Koichiro; Suzuki, Sanshiro
Corporate Source: Kansai Univ, Suita, Jpn
Source: Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 271-280
Publication Year: 1985
Abstract: The response of a multi-degree-of-freedom poly-linear hysteretic system subjected to non-stationary random excitation is examined by equivalent linearization. An ordinary differential equation for the covariance response is readily determined. Using this response, a technique of aseismic safety analysis of a multi-degree-of-freedom structural system with strength deterioration is presented. Numerical examples are given to show the accuracy of the presented technique for the covariance response and the distribution of the safety probability of each story and the reliability of the whole structural system. (Author abstract) 9 refs.

Title: **HYDROELASTIC RANDOM RESPONSE AND DECOMPOSED MATRIX PERTURBATION METHOD OF PLATFORMS.**
Author: Zhang, Xi-Cheng; Wu, You-Sheng; Xu, Dao-Lin
Corporate Source: Dalian Inst of Technology, Dalian, China
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 363-368 Publication Year: 1988
Abstract: This paper provides a study, as a part of the integrity monitoring of offshore platform structures, concentrating on the dynamic response analysis for frequency domain of platforms in irregular waves by the linearized Morison hydroelastic equation. Directional wave spectra and cross-spectral density function between velocities of wave flow are considered. This paper also carries out the sensitivity analysis of structures to the significant uncertainties rising from sea organism and dead loading. Based on the frequency domain, a decomposed matrix perturbation method is presented; it saves the CPU time. A simplified model of a jacket is applied to calculation. The differences are illustrated with figures. (Author abstract) 4 refs.

Title: **PROBABILISTIC DESIGN - A FREQUENCY-DOMAIN RESPONSE METHODOLOGY.**
Author: Spillane, M. W.; Leverette, S. J.
Corporate Source: Offshore Systems & Analysis Corp, Richmond, TX, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 157-164 Publication Year: 1988
Abstract: A unique frequency-domain probabilistic design methodology is introduced, using line tension in a deep-water mooring as a design example. The uniqueness of the analysis procedure is derived from the use of empirical orthogonal functions (EOF) to synthesize the structure of parameterized input spectra and transfer functions. The procedure may be applied quite generally to a wide range of systems (vessels or compliant structures) that are subject to stochastic forcing by the environment. (Author abstract) 3 refs.

EARTHQUAKE ANALYSIS STATE OF PRACTICE:

Title: Nonstationary response analysis of offshore guyed tower subjected to earthquake loading.
Author: Ryu, Chung-Son; Yun, Chung-Bang
Corporate Source: Dongshin Univ, Naju, South Korea
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 235-242 Publication Year: 1992
Abstract: A method for nonstationary response analysis of an offshore guyed tower subjected to earthquake ground excitations is presented. The equation of motion is formulated for the horizontal motion of the guyed tower using a multi-degree of freedom model. The nonstationarity of the earthquake excitation is modeled by imposing a time varying envelope function onto a stationary random component. The nonlinear hydrodamping and the nonlinear restoring force of the guyline system are included in analysis with the aid of the equivalent linearizations. The nonstationary responses are obtained in terms of time dependent variance functions. By taking the auto-correlation function of the nonstationary ground acceleration in terms of complex exponential functions of time, an analytical procedure is developed for computing time varying variances of the tower responses. Expected maximum responses are evaluated thereafter. Example analysis indicates that the maximum responses estimated by considering nonstationary effect properly are significantly less than those obtained by the conventional spectral method based on the stationary assumption. (Author abstract) 13 Refs.

Title: Seafloor seismic network offshore southern California.
Author: Smith, Charles E.
Corporate Source: Minerals Management Service, Herndon, VA, USA
Source: NIST Special Publication n 820 Sep 1991. Publ by Natl Inst of Standards & Technology, c/o US Department of Commerce, Gaithersburg, MD, USA. p 87-97 Publication Year: 1991
Abstract: Designing offshore facilities to resist earthquakes pose perplexities that differ from those problems encountered in designing similar facilities for other environmental loads, such as winds, waves, currents, and ice floes. Normal coperational loads occur frequently over the lifetime of a structure, and therefore, design information can be accumulated quickly. Because of this, design codes and other standards of practice have evolved fairly rapidly to address these normal types of loads. In contrast, earthquakes are rare events and data on the response of seafloor sediments to earthquake-induced ground motions are scarce, thereby introducing significant incertainty for analyzing seismic-hazard aspects of offshore operations. To reduce this uncertainty, a research program has been undertaken by the Minerals Management Service, jointly with industry, to develop and deploy instrumentation to measure seafloor seismic motions and to gather data to assess the effects of large seismic events predicted for the future. The primary objectives of this paper are to present an overview of the Seafloor Earthquake Measurement System (SEMS) Program and to describe the installation of a SEMS network offshore southern California. (Author abstract) 9 Refs.

Title: Probabilistic multi-objective optimal design of seismic-resistant braced steel frames using ARMA models.
Author: Takewaki, I.; Conte, J. P.; Mahin, S. A.; Pister, K. S.
Corporate Source: Univ of California, Berkeley, CA, USA
Source: Computers and Structures v 41 n 4 1991 p 687-707 Publication Year: 1991
Abstract: The objective of this paper is to develop a probabilistic multi-objective optimal design method for concentrically braced steel frames, including the design earthquake via a dynamic ARMA (Auto-Regressive Moving Average) model. The features of this design method are; (i) to make it possible to incorporate inherent uncertain features of design earthquakes into the design process itself through the dynamic ARMA model, (ii) to provide a simplified design formula for a preliminary design of concentrically braced steel frames based upon the concept of decompossed stiffness design, and (iii) to facilitate the formulation of a new probabilistic multi-objective optimal design problem aimed at finding the design with the minimum level of designer's dissatisfaction. In this optimal design problem, constraints and objectives are handled in a unified manner after a feasible design is obtained. Two design

examples are presented to demonstrate the validity of this design method. Finally, the generality and practicality of the design method are assessed. (Author abstract) 45 Refs.

Title: Multiple level dynamic substructure analysis.
Author: Yuan, Mingwu; Xiong, Shanji; Chen, Xiaohong
Corporate Source: Peking Univ, Beijing, China
Source: Engineering Computations (Swansea, Wales) v 8 n 3 Sep 1991 p 231-244 Publication Year: 1991
Abstract: An exact multiple-level dynamic substructure technique was developed by a combination of WYD algorithm and static multiple-level substructuring technique. This method is essentially different from the traditional mode component synthesis. The eigenvalues and eigenvectors created by the method are the eigenpairs for the whole structure and not for the components of structure. On the other hand, the dynamic response by using mode superposition can also be implemented in substructure level. This algorithm actually is an exact substructuring technique which means that substructuring itself did not introduce any additional error except the round-off when a structure was split into some arbitrary subdomains and the error of WYD or mode superposition themselves. It is no longer necessary to assume any connective condition on the interface between substructures. This method makes the capacity of dynamic analysis of a structural analysis program unlimited. It is especially attractive for the programs on microcomputers. Of course, the method leads to a frequent I/O for a subsequent search of the files from each substructure. It is time consuming compared to the mode component synthesis. But the potential still exists to improve the efficiency by using parallel computation on concurrent computers. In this paper the theory and procedure of the algorithm are presented. (Author abstract) 8 Refs.

Title: Computational tool for evaluation of seismic performance of reinforced concrete buildings.
Author: Kunnath, S. K.; Reinhorn, A. M.; Abel, J. F.
Corporate Source: State Univ of New York at Buffalo, Amherst, NY, USA
Source: Computers and Structures v 41 n 1 1991 p 157-173 Publication Year: 1991
Abstract: A special-purpose computational tool is developed to evaluate the inelastic seismic response of reinforced concrete buildings. A macromodel approach is used to analyze a discretized building composed of parallel frame-wall systems interconnected by transverse elements. The macro-behavioral models include the essential hysteretic characteristics of reinforced concrete sections and also account for the effects of distributed plasticity. All developments are incorporated into a computer code, IDARC. The program performs a series of tasks to enable a complete evaluation of the structural system: (a) monotonic analysis to establish the collapse mechanism and base shear capacity of the structure; (b) quasi-static cyclic analysis under force or deformation control; (c) transient dynamic analysis under horizontal and vertical seismic excitations; (d) reduction of the response quantities to damage indices so that a physical interpretation of the response is possible. The program is built around two graphical interfaces: one for preprocessing of structural and loading data; and the other for visualization of structural damage following the seismic analysis. The program can serve as an invaluable tool in estimating the seismic performance of existing reinforced concrete buildings and for designing new structures within acceptable levels of damage. (Author abstract) 14 Refs.

Title: U.S. seismic steel codes.
Author: Popov, Egor P.
Corporate Source: Univ of California, Berkeley, CA, USA
Source: Engineering Journal of the American Institute of Steel Construction v 28 n 3 3rd Quarter 1991 p 119-128 Publication Year: 1991
Abstract: Major changes and recomendations for the seismic design of steel buildings were recently introduced into several U.S. design codes and specifications. Many of the changes deal primarily with material and detailing requirements. However, these problems cannot be separated from the underlying issues of lateral seismic loads. Therefore in this paper a broader point of view is adopted, and a brief discussion of relevant findings from seismology and geotechnical engineering is presented first. A general discussion of structural code developments and the relationships among the lateral load requirements in different codes follows. Specific issues pertaining to the seismic design of structures are

then brought in. Limitations of the currently dominant elastic methods of analysis and design for seismic resistant structures are critically examined. The paper concludes with suggestions for future research. 18 Refs.

Title: 3-D nonlinear seismic behavior of cable-stayed bridges.
Author: Abdel-Ghaffar, Ahmed M.; Nazmy, Aly S.
Corporate Source: Univ of Southern California, Los Angeles, CA, USA
Source: Journal of Structural Engineering v 117 n 11 Nov 1991 p 3456-3476 Publication Year: 1991
Abstract: The dynamic behavior of three-dimensional (3-D) long-span cable-stayed bridges under seismic loadings is studied. The cases of synchronous and nonsychronous support motions due to seismic excitations of these flexible structures are considered: furthermore, effects of the nondispersive traveling seismic wave on the bridge response are studied. Different sources of nonlinearity for such bridges are included in the analysis. Nonlinearities can be due to: (1) Changes of geometry of the whole bridge due to its large deformations, including changes in the geometry of the cables due to tension changes (known as the sag effect); and (2) axial force and bending moment interaction in the bridge tower as well as the girder elements. A tangent stiffness iterative procedure is used in the analysis to capture the nonlinear seismic response. Numerical examples are presented in which a comparison is made between a linear earthquake-response analysis (based on the utilization of the tangent stiffness matrix of the bridge at the dead-load deformed state) and a nonlinear earthquake-response analysis using the step-by-step integration procedure. In these examples, two models having center (or effective) spans of 1,100 ft (335.5 m) and of 2,200 ft (671 m) are studied; this range covers both present and future designs. The study sheds some light on the salient features of the seismic analysis and design of these long contemporary bridges. (Author abstract) 10 Refs.

Title: Development of design spectra for actively controlled wall-frame buildings.
Author: Wang, Y. P.; Reinhorn, A. M.; Soong, T. T.
Corporate Source: Chung-Hwa Polytech Inst, Hsin-chu, Taiwan
Source: Journal of Engineering Mechanics v 118 n 6 Jun 1992 p 1201-1220 Publication Year: 1992
Abstract: Preliminary design of a structure controlled by active bracing or mass dampers to withstand seismic motion or high winds requires rapid means of analysis and estimates. A response spectrum technique applied to simplified models of complex structures can provide such means. The spectra required for preliminary evaluation of a controlled structure include, besides the usual response parameters (displacement, acceleration, base shear), the control resource parameters, such as the size and capacity of hydraulic actuators: power requirements: and energy consumption. This paper presents an approach to develop design spectra based on the analogy of behavior of wall frame structures with the behavior of shear-flexural cantilever beams treated by the continuum approach. This approach is verified numerically and experimentally using simplified parameters representing the structures. The same approach is further used to develop typical response and resource spectra using simulated ground motions according to a current seismic code. Several recommendations on efficiency of control of wall-frame structures are presented using information from the design spectra developed herein. (Author abstract) 19 Refs.

WAVE DYNAMICS

Title: WAVE PHASE EFFECTS ON DYNAMIC RESPONSE OF OFFSHORE PLATFORMS.
Author: Hsieh, C. C.; Kareem, A.; Williams, A. N.
Corporate Source: Univ of Houston, Houston, TX, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 327-332 Publication Year: 1988
Abstract: The influence of wave phase effects on the dynamic response of offshore platforms subjected to random waves is investigated. A typical platform may experience a reduction in both global and local forces due to the difference in wave phase resulting from the spacing between members in the direction of wave propagation. A frequency domain analysis was performed to investigate the effectiveness of wave cancellation for both deterministic and random sea-states utilizing multiple-leg configurations representing the waterline geometries of a typical offshore platform. The influence of hydrodynamic damping resulting from the relative velocity term in the description of the hydrodynamic loading was included. The numerical results indicate that the reduction in wave loading due to phase effects may be sizable and that the modifications to the wave surface profile due to the above effects are relatively less significant. (Author abstract) 12 refs.

Title: BIOFOULING AND MORISON EQUATION COEFFICIENTS.
Author: Nath, J. H.
Corporate Source: Oregon State Univ, Corvallis, OR, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 55-64 Publication Year: 1988
Abstract: The mean wave force coefficients from the Morison equation for cylinders with 28 surface roughnesses, including biofouling, are presented. Tests were performed at Oregon State University with steady flow and large laboratory periodic waves on vertical and horizontal cylinders. The open literature deals mostly with smooth and sand roughened cylinders, so this information on marine grown roughnesses was generated. In addition, some of the effects from cylinder orientation, periodic vs. non-periodic waves, a non-zero current, the wave parameters, and laboratory vs. field measurements, are discussed. (Author abstract) 25 refs.

Title: EXPERIMENTAL STUDIES ON THE RESPONSE OF AN IDEALIZED OFFSHORE STRUCTURE.
Author: Jain, A. K.
Corporate Source: Indian Inst of Technology, New Delhi, India
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 39-46 Publication Year: 1988
Abstract: Laboratory measurements of the deck displacement of a fixed offshore structure in regular waves are compared with predictions from the Morison equation results. The effects of variable submergence, associated with the passage of waves and fluid-structure interaction in the drag force are studied on the dynamic behaviour of offshore structures. An idealized structure has been analysed under the action of regular waves. The equations of motion are transformed into modal coordinates and then solved in iterative frequency domain. Results show that the Morison equation, with Airy's linear wave theory modified as suggested by Chakrabarti for taking variable submergence into account and constant drag and inertia coefficients of 1. 2 and 2. 0 respectively, provides good agreement with the deck displacement values measured over a complete steady-state cycle. (Edited author abstract) 11 refs.

Title: CONTROL OF DYNAMIC RESPONSES OF TOWER LIKE OFFSHORE STRUCTURES IN WAVES.
Author: Yoshida, K.; Suzuki, H.; Oka, N.
Corporate Source: Univ of Tokyo, Tokyo, Jpn
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 249-256 Publication Year: 1988

Abstract: This paper presents a preliminary attempt to control the dynamic response of a tower-like offshore structure subjected to regular waves. The structures are modeled in two ways. One is a vertical rigid pipe supported at the lower end by a pin joint. The other is a vertical flexible pipe fixed at the lower end. The formation of the optimal control shows that the control consists of a feedback control and a feedforward control based on the disturbance. In this research, two types of feedforward control are employed apart from the optimality. One is to compensate the entire wave forces acting on the structure. The other is on-off control to compensate the principal Fourier component of the wave forces by using the three states of the thruster, forward, stop and backward. The displacement and deformation of the structures were measured by an ultrasonic measurement system. (Edited author abstract) 15 refs.

Title: RANDOM EXTREME WAVE ANALYSIS OF DEEPWATER STRUCTURES.
Author: Chen, C. Y.; Armbrust, S.; Llorente, C.
Corporate Source: Hudson Engineering Corp, Houston, TX, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 165-171 Publication Year: 1988
Abstract: This paper first reviews various random wave analysis procedures for designing deepwater structures under an extreme seastate. The random wave analysis procedures suitable for fixed stiff platforms and compliant towers are discussed. The random wave analysis procedures are then applied to a 1350 ft. water depth fixed platform. The reduction in design force levels due to random waves is indicated by comparing with the conventional regular wave analysis approach. The second harmonic effects due to waves can be easily identified through the dynamic response spectrum which has two peaks occurring at the peak frequency of the input wave spectrum and the natural frequency of the structure. The study also shows that the expected extreme value estimated based on the upcrossing approach agrees well with the snapshot peak response derived from a wave record containing an extreme wave height. (Author abstract) 23 refs.

Title: STOCHASTIC APPROACH TO LIFE EXPECTANCY OF OFF-SHORE STRUCTURES UNDER RANDOM WAVE FORCE.
Author: Iyer, K. S. S.; Ukidave, V. H.
Corporate Source: Coll of Military Engineering, Poona, India
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 607-612 Publication Year: 1985
Abstract: The offshore structure experiences random loading conditions due to variations resulting from wave motions, weather conditions and ship movements. Therefore, the reliability has to be established considering the response of structure subjected to random waves. The offshore structure designed for a particular waveforce will be subjected to wave forces of larger magnitude. The time dependent cumulative damage to an offshore structure due to repetitive occurrence of waves of significant height has been studied and the probability of failure occurring in the structure due to wave conditions has been obtained. (Author abstract) 6 refs.

Title: VIBRATION CONTROL OF A CYLINDRICAL OFF-SHORE STRUCTURE.
Author: Yang, C. S.; Shimogo, T.
Corporate Source: Keio Univ, Yokohama, Jpn
Source: ASME Des Eng Div Publ DE v 7. Publ by ASME, New York, NY, USA p 157-164 Publication Year: 1987
Abstract: In the design of an off-shore tower-like structure, which consists of two pipe sections assembled with universal joints and attached to a buried sinker block on the sea bed, enough buoyancy can be provided to keep the structure vertical. It is desirable to reduce both the structural oscillations due to random wave excitation forces and the reaction forces acting on the sinker block. Assuming that the motion of the structure is two-dimensional in the vertical plane, these design requirements can be effectively satisfied if appropriate restoring and damping coefficients are provided at the joints. In this study, we employed the statistical equivalent linearization technique to linearize a non-linear fluid drag force, and applied a sub-optimal control theory with control structural constraints to obtain the optimal

stiffness and damping coefficients required at the joints after a trade-off between the rms values of the top displacement of structure and the rms values of control moments at the joints. (Edited author abstract) 7 refs.

Title: Uncertain parameter effects on dynamic response of offshore structure.
Author: Kawano, Kenji; Venkataramana, K.
Corporate Source: Kagoshima Univ, Kagoshima, Jpn
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 682-686 Publication Year: 1992
Abstract: The dynamic characteristics involved in the loading process in the ocean environment and also the characteristics of the subsoil are random in nature, and are difficult for exact evaluations of the offshore structure response. The uncertainties associated with them can be considered in the response analysis by stochastic description. In this study, emphasis is placed upon examination of uncertainty effects for jacket-type offshore structure. Sea waves are represented with the Bretschneider's power spectrum and the wave force is evaluated with the modified Morison equation. The dynamic equations of motion are derived by the substructure method and the governing equations involving the uncertainties are formulated by means of the perturbation method. The dynamic response analysis is carried out using the frequency-domain random-vibration approach. The response quantities are evaluated using the principles of the first passage probabilities across specific barriers. (Author abstract)

Title: Two basic concepts in offshore engineering.
Author: Hahn, Guillermo D.
Corporate Source: Vanderbilt Univ, Nashville, TN, USA
Source: Proceedings of Engineering Mechanics. Publ by ASCE, New York, NY, USA. p 188-191 Publication Year: 1992
Abstract: Two concepts are developed which lead to an improved understanding of the characteristics of the wave forces that act on deep-water, jacket-type offshore structures. The first concept applies to the inertia component of the wave loading; the second concept relates to the associated drag force component. These concepts further contribute to simplify the analysis and understanding of the dynamic response of such structures to wave excitations, and are of practical usefulness. (Author abstract) 1 Ref.

Title: STOCHASTIC FATIGUE OF NONLINEAR OFFSHORE STRUCTURAL SYSTEMS.
Author: Haldar, A.; Kanegaonkar, H. B.
Corporate Source: Georgia Institute of Technology, Atlanta, GA, USA
Source: Publ by Inst for Risk Research, Univ of Waterloo, Waterloo, Ont, Can v 1, p 87-94 Publication Year: 1987
Abstract: Nonlinearities in the wave loading due to nonlinear wave kinematics and free surface fluctuation are considered for a jacket-type platform. The first four moments of the response are estimated using the mean square estimation technique via conditional distribution. Nonlinear stiffness is considered for a guyed tower system. Approximating the loading by the ARMA process, Ito stochastic differential equations for the response moments are solved up to fourth order where the system of equations is closed by neglecting the fifth and higher order cumulants. The response moments are considered to be a mixture of Gaussian and non-Gaussian distributions. By mapping a Gaussian process into this response process, the expected rate of positive crossings is estimated, leading to the probability density of the peaks. Palmgren-Miner's hypothesis for fatigue damage accumulation is used. It is shown that the conventional method is unconservative when the response distribution is leptokurtic. (Edited author abstract) 13 refs.

Title: INTERACTION OF STEEL JACKET PLATFORM WITH SURROUNDING SOIL.
Author: Abduljauwad, S. N.; Sture, S.
Corporate Source: King Fahd Univ of Petroleum & Minerals, Dhahran, Saudi Arabia
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 355-361 Publication Year: 1988
Abstract: Offshore platforms have been developed in a variety of sizes, configurations and degrees of complexity, and there exists no unified procedure for their design and analysis. The purpose of this

investigation was to study the behavior of a fixed offshore jacket platform subjected to wind and wave loading. The purpose of this study was to present a methodology for performing analysis of the complete structure-pile-soil system; to demonstrate its application; and to compare the results obtained in this integrated analysis to those found by means of a conventional analysis procedure. This study also presents a methodology, based on the finite element method, for developing lateral soil resistance-pile displacement relationships for a single pile subjected to cyclic lateral loading. Undrained soil strengths were used in conjunction with the total stress analysis. (Edited author abstract) 10 refs.

Title: DESIGN LOAD METHOD FOR OFFSHORE STRUCTURES BASED UPON THE JOINT PROBABILITY OF ENVIRONMENTAL PARAMETERS.
Author: Madsen, M. N.; Nielsen, J. B.; Klinting, P.; Knudsen, J.
Corporate Source: Danish Hydraulic Inst, Horsholm, Den
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 75-80 Publication Year: 1988
Abstract: This paper describes a procedure for determining hydrodynamic design loads on offshore structures considering the joint probability of occurrence of the metocean phenomena (wind, waves, current, etc.). By accounting in a realistic manner for the joint probability, the new procedure can produce results which are of more uniform quality than more traditional methods. The procedure is based on the utilization of simultaneous time series of the environmental parameters. Databases containing this type of information are today available for some important offshore areas. (Edited author abstract) 9 refs.

Title: WAVE LOAD ESTIMATION IN DEEP WATER AS AFFECTED BY WAVE PERIOD ERRORS.
Author: Marshall, A. L.
Corporate Source: Sunderland Polytechnic, Sunderland, Engl
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 2. Publ by ASME, New York, NY, USA p 33-37 Publication Year: 1988
Abstract: The paper develops a procedure for estimating the sensitivity of deep water wave load calculation to variations or errors in wave period. Based on Stokes Fifth Order wave theory, it is shown that drag, inertia and total loads are disproportionately altered by such variations or errors. Comparison is made with a similar theory based on linear wave theory. The paper also incorporates a simplified deep water version of Skjelbreia and Hendrickson's solution of Fifth Order wave theory. (Author abstract) 9 refs.

Title: WAVE ATTENUATION, MUDSLIDE, AND STRUCTURAL ANALYSES FOR MAIN PASS/MISSISSIPPI DELTA CAISSON.
Author: Clukey, E. C.; Maller, A. V.; Murff, J. D.; Miller, M. C.; Goodwin,
Corporate Source: Exxon Production Research Co, Houston, TX, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 327-334 Publication Year: 1988
Abstract: The environmental loads associated with the design of a caisson-type structure in a mudslide region are largely dependent upon the interaction of waves with the seabottom. This wave-seabottom interaction results in both an attenuation of wave energy as the waves propagate over the soft soils, as well as the generation of mudslide loads at the platform site. These effects were investigated for the design of a 14-ft diameter caisson in the Main Pass region of the Mississippi Delta. The proposed structure was planned for installation in about 100-ft water depth in very soft underconsolidated soils, typical of the Mississippi Delta region. The potential for wave attenuation was investigated analytically by considering the change in response spectra from deep water to the platform site. Regional soil conditions were used for this part of the investigation. The results obtained from the analytical modeling were in good agreement with field measurements obtained from another part of the Delta. (Edited author abstract) 16 refs.

Title: STRUCTURAL DYNAMICS APPROACH ON THE WAVE INDUCED STRUCTURAL RESPONSES OF SEMISUBMERSIBLE TYPE PLATFORMS.

Author: de O. Goncalves, Franklin
Corporate Source: Det Norske Veritas, Rio de Janerio, Braz
Source: Publ by Pentech Press, London, Engl p 352-369 Publication Year: 1986
Abstract: Wave induced stresses in semisubmersible platforms usually are predicted based on the assumption that they are sufficiently stiff to be considered as a rigid body. However, as the demand for higher deck payload in semisubmersibles increases coupled with the necessity to operate in harsher environments, and also in the case that they are used as production platforms for very deep waters, semisubmersibles will tend in the future to become larger in scale and probably structurally less stiff. Therefore the rigid body assumption will no longer hold true and the traditional quasi-static approach will not suffice. Due to this fact a structural dynamics approach to the wave induced stress analysis of semisubmersibles has been developed. (Edited author abstract) 21 refs.

Title: INTERACTION OF SOIL-PILE SYSTEMS SUBJECTED TO DYNAMIC EXCITATION.
Author: Paganelli, Leopoldo M.; Ebecken, N. F. F.
Corporate Source: Petrobras, Braz
Source: Publ by Pentech Press, London, Engl p 205-224 Publication Year: 1986
Abstract: Offshore structures supported by piles are often subjected to dynamic loads such as wave, wind, rotating machines and sometimes earthquake if the site is an area of seismic risk. An analytical theory, developed by Noyak and Nogami, for linear viscoelastic materials was used in the development of an experimental computer program in order to compare that solution with the lumped mass model, studying the differences of behavior in function of the excitation frequency. This analytical theory assumes linear viscoelastic soil, and pile of circular cross-section vibrating horizontally in a vertical plane. 16 refs.

Title: MEASURED FATIGUE STRESS RESPONSE OF NORTH SEA JACKET PLATFORMS.
Author: Heavner, John W.; Thuestad, Thore; Langen, Ivar; Syvertsen, Kare
Corporate Source: SINTEF, Trondheim, Norw
Source: v 3. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 719-723 Publication Year: 1985
Abstract: Strain response, wave loading and estimated fatigue damage for two North Sea steel jackets are evaluated from full scale measurements and compared with common theoretical models. The strain response is found to be quasi-static, wide banded and non-Gaussian. The non-linear nature of the strain response is shown to be indicative of Morison type drag loading. Strain cycle distributions are compared with fitted Weibull and Rayleigh distributions. The comparison is found to be reasonable for the Weibull distribution. Fatigue damage in short term sea states estimated for the sample strain, the fitted Weibull and the Rayleigh distributions demonstrates the conservative nature of the Rayleigh model. Long term fatigue damage estimates based on measurements indicates that the single wave deterministic analysis used in design is conservative. (Author abstract) 4 refs.

Title: Parametric influence on extreme dynamic response of drag-dominated platforms.
Author: Karunakaran, D.; Leira, B. J.; Haver, S.; Moan, T.
Corporate Source: SINTEF Structural Engineering, Trondheim, Norw
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 463-471 Publication Year: 1992
Abstract: The wave loading on slender offshore platforms is often drag-dominated and hence the loading is of a non-Gaussian nature. Furthermore, the response will exhibit significant dynamic behaviour. As a result, the calculated extreme response of such structures will be highly sensitive to a number of parameters and modelling aspects. In particular, models related to the frequency distribution of the wave loading relative to the natural frequency of the structure will be important. Factors affecting the dynamic amplification level, e.g. damping and soil stiffness, will also have a pronounced effect on the response level. In this paper the sensitivity of the calculated extreme response is investigated for two different slender platforms, a jacket and a jack-up, both of which are designed for moderately deep waters. The main emphasis is put on studying the influence from uncertainties related to the modelling of the wave spectral density. The response variation for different spectral density models is compared for the cases

with two different damping levels and two different first natural periods, for both structures. (Author abstract)

Title: **Stress-response of offshore structures by equivalent polynomial expansion techniques.**
Author: Sigurdsson, G.; Nielsen, S. R. K.
Corporate Source: Univ of Aalborg, Aalborg, Den
Source: Proc First 90 Eur Offshore Mech Symp. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 134-140 Publication Year: 1990
Abstract: This paper concerns an investigation of the effects of nonlinearity of drag loading on offshore structures excited by 2D wave fields, where the nonlinear term in the Morison equation is replaced by an equivalent cubic expansion. The equivalent cubic expansion coefficients for the equivalent drag model are obtained using the least mean square procedure. Numerical results are given. The displacement response and the stress response processes obtained using the above loading model are compared with simulation results and those obtained from equivalent linearization of the drag term. (Author abstract) 8 Refs.

Title: **DYNAMIC RESPONSE OF OFFSHORE STRUCTURES TO OCEAN WAVES.**
Author: Ishida, Hajime
Corporate Source: Kanazawa Univ, Kanazawa, Jpn
Source: v 2. Publ by Int Assoc for Structural Safety & Reliability, New York, NY, USA p 602-606 Publication Year: 1985
Abstract: As the basic study to construct the marine structures in thorough consideration of safety, this paper has dealt with the dynamic response of pile to ocean waves by means of numerical computations and laboratory experiments. The kinds of waves used here are the small amplitude waves, Stokes waves, hyperbolic waves and irregular waves. The wave forces were estimated from Morison's formula. For calculations of the displacement of pile, the transfer matrix method and the structural-property one were used. As the results, the displacements can be estimated adequately from the waves, and the resonance which makes structure dangerous is caused by the waves having the period of integer times natural period, the breaking waves, or some irregular waves. (Author abstract) 4 refs.

Title: **INFLUENCE OF CURRENT AND VARIABLE SUBMERGENCE ON DYNAMIC RESPONSE OF OFFSHORE STRUCTURES.**
Author: Jain, A. K.
Corporate Source: Indian Inst of Technology, New Delhi, India
Source: American Society of Mechanical Engineers, Petroleum Division (Publication) PD v 12. Publ by ASME, New York, NY, USA p 47-54 Publication Year: 1988
Abstract: The influence of current and variable submergence on the wave induced dynamic response of a slender offshore tower is investigated. The dynamic response of the structure is obtained by a frequency domain iterative procedure which can efficiently handle the nonlinearity due to the relative velocity-squared drag term. The numerical studies show the influence of the current and variable submergence on the frequency composition of hydrodynamic loading and on the response under both resonating and non-resonating conditions. (Edited author abstract) 11 refs.

Title: **CONTROL OF DYNAMIC RESPONSES OF TOWER LIKE OFFSHORE STRUCTURES IN WAVES.**
Author: Yoshida, K.; Suzuki, H.; Oka, N.
Corporate Source: Univ of Tokyo, Tokyo, Jpn
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 249-256 Publication Year: 1988
Abstract: This paper presents a preliminary attempt to control the dynamic response of a tower-like offshore structure subjected to regular waves. The structures are modeled in two ways. One is a vertical rigid pipe supported at the lower end by a pin joint. The other is a vertical flexible pipe fixed at the lower end. The formation of the optimal control shows that the control consists of a feedback control and a feedforward control based on the disturbance. In this research, two types of feedforward control are employed apart from the optimality. One is to compensate the entire wave forces acting on the structure.

The other is on-off control to compensate the principal Fourier component of the wave forces by using the three states of the thruster, forward, stop and backward. The displacement and deformation of the structures were measured by an ultrasonic measurement system. (Edited author abstract) 15 refs.

Title: RANDOM EXTREME WAVE ANALYSIS OF DEEPWATER STRUCTURES.
Author: Chen, C. Y.; Armbrust, S.; Llorente, C.
Corporate Source: Hudson Engineering Corp, Houston, TX, USA
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 165-171 Publication Year: 1988
Abstract: This paper first reviews various random wave analysis procedures for designing deepwater structures under an extreme seastate. The random wave analysis procedures suitable for fixed stiff platforms and compliant towers are discussed. The random wave analysis procedures are then applied to a 1350 ft. water depth fixed platform. The reduction in design force levels due to random waves is indicated by comparing with the conventional regular wave analysis approach. The second harmonic effects due to waves can be easily identified through the dynamic response spectrum which has two peaks occurring at the peak frequency of the input wave spectrum and the natural frequency of the structure. The study also shows that the expected extreme value estimated based on the upcrossing approach agrees well with the snapshot peak response derived from a wave record containing an extreme wave height. (Author abstract) 23 refs.

Title: VIBRATION TESTING AND DYNAMIC ANALYSIS OF OFFSHORE PLATFORM MODELS.
Author: Zhao, Yin; Chen, Zhongyi; Ke, Renqun
Corporate Source: Nanjing Hydraulic Research Inst, Nanjing, China
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium 7th v 1. Publ by ASME, New York, NY, USA p 85-89 Publication Year: 1988
Abstract: This paper describes the application of an experimental modal analysis technique to the models of a deepwater jacket fixed platform and a jack-up drilling platform with a foundation mat and presents the results of testing and analysis. Comparisons have been made with finite element analyses and other dynamic analyses. In addition, dynamic displacement responses of the model platforms have been measured when subjected to waves in a wave basin. (Author abstract) 6 refs.

Title: EXPERIMENTAL OBSERVATIONS ON OFFSHORE STRUCTURAL MODELS.
Author: Teixeira, Evandro Dessimoni; Roehl, Joao Luis Pascal
Corporate Source: Pontificia Univ Catolica, Rio de Janeiro, Braz
Source: Publ by Pentech Press, London, Engl p 805-816 Publication Year: 1986
Abstract: In the evaluation of the response of marine structures some aspects are highly involved, mainly due to difficulties in understanding the effects produced by the presence of water on the behavior of the structure, and vice-versa. Determination of the dynamic characteristics of the immersed structure and computation of wave forces on these systems are among this set of problems. This paper covers an experimental work to assist in the development of procedures and observations for the analysis of such structures. The tests comprised of four reduced size models made of plexiglass, initially immersed in still water and after in uniform sea waves. The main objectives were the measurement of the frequency, damping and added mass, and the observation of the forces and displacements of the model. 6 refs.

Title: INTERACTION OF SOIL-PILE SYSTEMS SUBJECTED TO DYNAMIC EXCITATION.
Author: Paganelli, Leopoldo M.; Ebecken, N. F. F.
Corporate Source: Petrobras, Braz
Source: Publ by Pentech Press, London, Engl p 205-224 Publication Year: 1986
Abstract: Offshore structures supported by piles are often subjected to dynamic loads such as wave, wind, rotating machines and sometimes earthquake if the site is an area of seismic risk. An analytical theory, developed by Noyak and Nogami, for linear viscoelastic materials was used in the development of an experimental computer program in order to compare that solution with the lumped mass model, studying

the differences of behavior in function of the excitation frequency. This analytical theory assumes linear viscoelastic soil, and pile of circular cross-section vibrating horizontally in a vertical plane. 16 refs.

Title: Parametric influence on extreme dynamic response of drag-dominated platforms.
Author: Karunakaran, D.; Leira, B. J.; Haver, S.; Moan, T.
Corporate Source: SINTEF Structural Engineering, Trondheim, Norw
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 463-471 Publication Year: 1992
Abstract: The wave loading on slender offshore platforms is often drag-dominated and hence the loading is of a non-Gaussian nature. Furthermore, the response will exhibit significant dynamic behaviour. As a result, the calculated extreme response of such structures will be highly sensitive to a number of parameters and modelling aspects. In particular, models related to the frequency distribution of the wave loading relative to the natural frequency of the structure will be important. Factors affecting the dynamic amplification level, e.g. damping and soil stiffness, will also have a pronounced effect on the response level. In this paper the sensitivity of the calculated extreme response is investigated for two different slender platforms, a jacket and a jack-up, both of which are designed for moderately deep waters. The main emphasis is put on studying the influence from uncertainties related to the modelling of the wave spectral density. The response variation for different spectral density models is compared for the cases with two different damping levels and two different first natural periods, for both structures. (Author abstract)

Title: Statistical distribution of frequency response in disordered periodic structures.
Author: Cai, G. Q.; Lin, Y. K.
Corporate Source: Florida Atlantic Univ, Boca Raton, FL, USA
Source: AIAA Journal v 30 n 5 May 1992 p 1400-1407 Publication Year: 1992
Abstract: A structure designed to be spatially periodic cannot be exactly periodic. The departure from exact periodicity is known as disorder, and it causes localization of normal modes and additional attenuation of wave motions in the structure not related to damping. The present investigation is directed at a possible adverse effect of disorder, namely, higher structural response near the point at which a dynamic excitation is applied than would occur in a perfectly periodic structure, thereby reducing structure safety and reliability. A systematic procedure is developed herein for the analysis of such an effect for a generic disordered periodic structure. In particular, the probability distribution of structural response is obtained by analysis and by Monte Carlo simulation, which is needed both for fundamental understanding of the effect and for predicting the reliability of a system. It is shown also that, given probability distributions of the disordered parameters of a structure, the mean and standard deviation of structural response can be obtained exactly for a damped randomly disordered periodic structure if the number of disordered cell units is not large and, approximately, if the number is large. Application of the procedure is illustrated by an example, and the results are compared with Monte Carlo simulations. (Author abstract) 36 Refs.

WAVE DYNAMICS

Title: Drag force on cylinders oscillating at small amplitude: a new model.
Author: Anaturk, A. R.; Tromans, P. S.; van Hazendonk, H. C.; Sluis, C. M.;
Corporate Source: Technical Univ of Delft, Delft, The Neth
Source: Journal of Offshore Mechanics and Arctic Engineering v 114 n 2 May 1992 p 91-103
Publication Year: 1992
Abstract: A new semi-empirical model is derived for calculations of hydrodynamic damping (or drag) forces on smooth or rough cylinders oscillating at small amplitude and high frequency. This model covers the attached flow regime where the conventional Morison's equation fails to simulate the drag forces acting on circular cylinders. It involves the calculation of the energy dissipated in the boundary layer on the cylinder surface. The empirical input data for the model are amplitude and phase of the wall shear stresses on the cylinder. The model is verified against fluid force measurements from a test tank and published data. These experiments were carried out at small-amplitude/high-frequency with smooth and rough circular cylinders. Data from the literature is also included for verification. Results from the present work can be used to estimate hydrodynamic damping forces for the analysis of jack-up response around resonant frequency. Other applications are dynamic behavior of jackets during installation, SALMs, TLPs and risers. (Author abstract) 24 Refs.

Title: Statistics of the extreme response of offshore structures.
Author: Tromans, P. S.; Hagemeijer, P. M.; Wassink, H. R.
Corporate Source: Shell Research B.V., Rijswijk, Neth
Source: Ocean Engineering (Pergamon) v 19 n 2 Mar 1992 p 161-181 Publication Year: 1992
Abstract: For a model structure standing in the ocean we have derived the probability density function of the extreme base shear, that is, the extreme value of the total horizontal force exerted by storm waves and currents, in a storm interval. The structure may consist of a single, slender column or a closely spaced group of columns while the random wave motion is narrowbanded and Gaussian. The effect of the variation in wetted surface, storm duration, current and the non-linearity of the Morison load equation are included. Though the analysis treats base shear force, the method could equally well be applied to other quasi-static, structural responses. The analytical probability density function is in good agreement with histograms derived from time domain simulations of extreme base shear on a single column and on a real structure. Application to the real structure involves the generation of a stick analogy: the structure is represented by a group of columns that has the same hydrodynamic area and volume. (Author abstract)

Title: New model for the kinematics of large ocean waves application as a design wave.
Author: Tromans, Peter S.; Anatruk, A. H. R.; Hagemeijer, Paul
Corporate Source: Shell Research B.V., Rijswijk, Neth
Source: Proc First Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 64-71 Publication Year: 1991
Abstract: The displacement of the ocean surface has been derived for the region around a wave crest in a random sea. The solution has a random part and a deterministic part. The latter provides an excellent, deterministic model for the surface elevation and water particle velocities and accelerations induced by extreme ocean waves. It is essentially a linear, broad-banded wave theory in which the component wavelets have amplitudes and phases such that the most probable extreme wave is obtained immediately. The formulation is equivalent to the extensively validated random wave theories, but it involves the simulation of just one wave rather than many hours of a sea state. Application of this theory to fluid load assessment may offer the realism of time domain simulation of random wave fields with the speed and convenience of deterministic analysis. Loads on some simple model structures calculated using the present theory, time domain simulation of random waves and Stokes V waves have been compared. The agreement between the present theory and the time domain simulations is excellent. (Author abstract) 15 Refs.

Title: Environmental forces in relation to structure design or assessment. A personal view towards integration of the various aspects involved.
Author: Vugts, J. H.
Corporate Source: Shell UK Exploration and Production
Source: Underwater Technology v 17 n 1 Spring 1991 p 3-9 Publication Year: 1991
Abstract: Environmental forces are discussed in connection with the overall problem of the design of new, or the assessment of existing, structures. It is essential to see environmental forces in this broader context and to integrate them with all the other aspects involved. 'Structures' refers here in general to all types and applications - floating and bottom supported, compliant and fixed. Where a discussion is aimed at a particular group, this is specifically identified, such as in the case of fixed space frame structures, to which special attention is paid. Offshore structures engineering has in many respects become a mature discipline. However, further developments are still necessary to fully benefit from the large increase in knowledge after so many years of research and development efforts, as well as from the accumulated experience with real platforms offshore. This is in the best interest of safety, the extended use of existing structures and the more economical design of new ones. The next major step is sometimes suggested to be structural reliability analysis, which would require both the loading and the structural models (describing generally elastic as well as rigid body properties) to become probabilistic. In general this is not yet considered to be within reach and is cautioned against. However, the underlying principle is warmly supported and the introduction of concepts which are firmly based on reliability analysis methods is a realistic goal to aim for. It is suggested that significant progress is this direction can be made by using loading models which incorporate the probabilistic features which reflect the random wave environment in a realistic manner, in combination with (nearly) deterministic structural models. Where such loading models are not already current practice, they should be introduced forthwith. This demands a truthful representation of random wave kinematics and realistic force coefficients, including the drag and inertia coefficients in Morison's equation for fixed structures. A perceived way forward is described and proposals are made on how to achieve this. (Author abstract) 28 Refs.

Title: Wave-current and fluid-structure interaction effects on the stochastic analysis of offshore structures.
Author: Karadeniz, H.
Corporate Source: Delft Univ of Technology, Delft, Neth
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 687-693 Publication Year: 1992
Abstract: This paper deals with an advanced stochastic analysis for deep water offshore structures under wave-current and structure-fluid interactions. Having described the random sea and transfer functions of water velocities and accelerations in a wave-current field, wave forces, hydrodynamic damping ratios and added masses are calculated on the base of using the Morison's equation. The drag force term of the Morison's equation is linearized by using an equivalent second-moment criterion and the relative water velocity concept. Then, attention is paid on the structural response analysis. It is emphasized that a proper and correct analysis can be fulfilled by considering the wave-current and fluid-structure interactions. (Author abstract)

Title: Wave loading on dynamic sensitive offshore structures.
Author: Gudmestad, O. T.; Spidsoe, N.; Karunakaran, D.
Corporate Source: STATOIL, Stavanger, Norw
Source: Proceedings of the International Offshore Mechanics and Arctic Engineering Symposium v 1 pt A. Publ by American Soc of Mechanical Engineers (ASME), New York, NY, USA. p 247-253
Publication Year: 1990
Abstract: Inclusion of nonlinear loading and response terms and the dynamic amplification of wave forces on dynamic sensitive offshore structures require a time domain stochastic analysis of the structure's response under the extreme wave condition. A description of how a proper analysis should be carried out for dynamic fixed platforms will be presented. The methodology will be demonstrated by examples of the importance of nonlinear terms, dynamic effects, wave kinematics model chosen and wave-current

interaction effects. Particular reference will be made to a deep water slender jacket structure. (Author abstract) 8 Refs.

Title: On some uncertainties in the modelling of ocean waves and their effects on tlp response.
Author: Haver, Sverre; Naivig, Birger J.
Corporate Source: Statoil, Stavanger, Norw
Source: Proc First 90 Eur Offshore Mech Symp. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 278-284 Publication Year: 1990
Abstract: The effects of uncertainties in the modelling of ocean waves is considered both with respect to fatigue and predicted extremes. A tension leg platform is adopted as the example structure and the platform is assumed to be located at Haltenbanken. A statistical model for the wave climate is presented and some uncertainties related to this model are identified. Various models for the wave spectrum are also considered. The adequacy of these models with respect to the actual structural quantity is indicated by also calculating the fatigue damage using mean measured spectra. (Author abstract) 11 Refs.

Title: Stress-response of offshore structures by equivalent polynomial expansion techniques.
Author: Sigurdsson, G.; Nielsen, S. R. K.
Corporate Source: Univ of Aalborg, Aalborg, Den
Source: Proc First 90 Eur Offshore Mech Symp. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 134-140 Publication Year: 1990
Abstract: This paper concerns an investigation of the effects of nonlinearity of drag loading on offshore structures excited by 2D wave fields, where the nonlinear term in the Morison equation is replaced by an equivalent cubic expansion. The equivalent cubic expansion coefficients for the equivalent drag model are obtained using the least mean square procedure. Numerical results are given. The displacement response and the stress response processes obtained using the above loading model are compared with simulation results and those obtained from equivalent linearization of the drag term. (Author abstract) 8 Refs.

Title: Forces on an inclined cylinder in regular waves.
Author: Li, Yu-cheng; Kang, Hai-gui; Fei, Qing-hau
Corporate Source: Dalian Univ of Technology, Dalian, China
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 417-428 Publication Year: 1992
Abstract: Through a series of model tests, the wave forces on horizontal and inclined circular cylinders were measured and analyzed. Based on Morison Equation and Stokes second order wave theory, the relationship between the hydrodynamic force coefficients with KC number and submerged water depth as well for horizontal cylinder were analyzed. Also the relationship between the hydrodynamic force coefficients with KC number, inclined angle and the effect of water free surface as well as the inclined cylinder were investigated. (Author abstract)

Title: Uncertain parameter effects on dynamic response of offshore structure.
Author: Kawano, Kenji; Venkataramana, K.
Corporate Source: Kagoshima Univ, Kagoshima, Jpn
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 682-686 Publication Year: 1992
Abstract: The dynamic characteristics involved in the loading process in the ocean environment and also the characteristics of the subsoil are random in nature, and are difficult for exact evaluations of the offshore structure response. The uncertainties associated with them can be considered in the response analysis by stochastic description. In this study, emphasis is placed upon examination of uncertainty effects for jacket-type offshore structure. Sea waves are represented with the Bretschneider's power spectrum and the wave force is evaluated with the modified Morison equation. The dynamic equations of motion are derived by the substructure method and the governing equations involving the uncertainties are formulated by means of the perturbation method. The dynamic response analysis is carried out using the frequency-domain random-vibration approach. The response quantities are evaluated using the principles of the first passage probabilities across specific barriers. (Author abstract)

Title: Effects of free surface fluctuation on response of offshore structures.
Author: Tung, Chi C.; Yang, Cheng H.
Corporate Source: North Carolina State Univ, Raleigh, NC, USA
Source: Proc First Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 183-193 Publication Year: 1991
Abstract: In this paper, effect of free surface fluctuation on dynamic response of offshore structures is studied. A single degree of freedom structure is subjected to a long-crested sinusoidal wave in water of arbitrary depth. Wave force is computed according to the Morison equation and wave induced fluid particle velocity and acceleration are modified to account for the effects of free surface fluctuation. The equation of motion is solved by an approximate analytical method in closed form. Results indicate that free surface fluctuation causes higher harmonic responses, produces bias in response, and its effects become important as water depth decreases. (Author abstract) 13 Refs.

Title: Nonlinear effects on wave loads acting on vertical cylinders with various cross-sections.
Author: Masudo, K.; Goto, S.; Nagai, T.; Fujisawa, Y.
Corporate Source: Nihon Univ, Tokyo, Jpn
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 518-525 Publication Year: 1992
Abstract: The objectives of the present paper are to develop a powerful numerical method for computing second-order wave loads on three-dimensional bodies and to clarify the effects of nonlinear wave diffraction. In this paper, second-order wave loads are computed from the radiated wave potential of double frequency and first-order scattered wave potential by applying the extended Haskind formula. The infinite integral over the free surface is evaluated by using Fresnel and Gaussian integrations. The second-order oscillating wave loads on circular, rectangular and elliptic vertical cylinders are computed systematically. The second-order effects on wave forces are evaluated from numerical results and summarized in the charts. (Author abstract)

Title: Two basic concepts in offshore engineering.
Author: Hahn, Guillermo D.
Corporate Source: Vanderbilt Univ, Nashville, TN, USA
Source: Proceedings of Engineering Mechanics. Publ by ASCE, New York, NY, USA. p 188-191 Publication Year: 1992
Abstract: Two concepts are developed which lead to an improved understanding of the characteristics of the wave forces that act on deep-water, jacket-type offshore structures. The first concept applies to the inertia component of the wave loading; the second concept relates to the associated drag force component. These concepts further contribute to simplify the analysis and understanding of the dynamic response of such structures to wave excitations, and are of practical usefulness. (Author abstract) 1 Ref.

Title: Hydrodynamic characteristics and seismic analysis of offshore cylindrical structure-fluid-soil systems.
Author: Bi, J. J.; Zhuang, H.; Weng, X. I.
Corporate Source: Tongji Univ, Shanghai, China
Source: Proc First Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 264-269 Publication Year: 1991
Abstract: An analytical method is developed to solve the coupled vibration problem of an elastic vertical circular cylindrical structure founded on homogeneous elastic soil with surrounding fluid. An improved mathematical model is adopted so that not only the structure-fluid interaction but also the influence of elasticity of soil are considered. The frequency-dependent radiation added mass and hydrodynamic radiation damping are expressed in closed form, and are compared with the generalized mass and damping respectively. Their variations with the exciting frequency are studied as well. A discussion of the influence of structure-fluid interaction and structure-soil interaction upon hydrodynamic loading is also given. (Author abstract) 10 Refs.

Title: Hydrodynamic characteristics and seismic analysis of offshore cylindrical structure-fluid-soil systems.

Author: Jia-ju, Bi; Hong, Zhuang; Xiu-ling, Weng
Corporate Source: Tongi Univ, Shangkai, China

Source: Journal of Hydrodynamics v 3 n 3 1991 p 79-87 Publication Year: 1991

Abstract: An analytical method is developed to solve the coupled vibration problem of a vertical elastic circular cylindrical structure founded on homogeneous elastic soil with surrounding fluid. An improved mathematical model is adopted by which not only the structure-fluid interaction but also the influence of soil are considered. The frequency-dependent radiation added mass and hydrodynamic radiation damping are expressed in closed form, and are compared with generalized mass and damping respectively. Their variations with the exciting frequency are studied as well. A discussion of the influences of structure fluid interaction and structure-soil interaction upon hydrodynamic loading is given. (Author abstract) 10 Refs.

MODEL TESTS

Title: Integrated program system for response analysis of offshore structures.
Author: Yoneya, Takuya; Arima, Toshiro; Tsutsui, Yasuharu
Corporate Source: Nippon Kaiji Kyokai, Tokyo, Jpn
Source: Proc First 90 Eur Offshore Mech Symp. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 141-152 Publication Year: 1990
Abstract: This paper presents an outline of a comprehensive program system developed by the authors, which includes many functions for response analysis of a wide variety of offshore structures. Typical calculated results by this system are also presented being compared with field measurements and model test data as well as calculations by the other analysis programs. Through the comparative calculations the validity and flexibility applicability of the system have been confirmed and the results presented here may be valuable and useful for the designers and researchers of offshore structures. (Author abstract) 34 Refs.

SOIL STRUCTURE INTERACTION:

Title: Displacement and soil-structure interaction under dynamic and cyclic loading.
Author: Gazetas, George
Corporate Source: Mobil North Sea Ltd
Source: Deformation of Soils and Displacements of Structures X ECSMFE Proceedings of the International Conference on Soil Mechanics and Foundation Engineering v 3. Publ by A.A. Balkema, Rotterdam, Neth. p 1091-1104 Publication Year: 1991
Abstract: Eleven papers dealing with various aspects of the problem are reviewed. The discussion focuses on: vibration theories and their validation; analysis of porewater pressure buildup during strong offshore and seismic loading; and lateral earth pressures and bearing capacity under seismic excitation. Some recent developments in these areas are highlighted, while questions are raised and a few topics for further research are suggested. (Author abstract) 44 Refs.

Title: Soil resistance at high deformation rates. A 3-D approach.
Author: Stefess, H.; Van Der Graaf, H. J.; Van Ophem, J. I. M.
Corporate Source: Ministry of Transport and Public Works, Delft, Neth
Source: Deformation of Soils and Displacements of Structures X ECSMFE Proceedings of the International Conference on Soil Mechanics and Foundation Engineering v 1. Publ by A.A. Balkema, Rotterdam, Neth. p 265-268 Publication Year: 1991
Abstract: A study is performed to determine the soil resistance encountered by a structure penetrating into saturated sands. The forces exerted on the structure are governed by pore water under pressure, generated by shear-induced volume-expansion (dilatancy). Both an analytical approach and model tests were performed in order to determine the forces on the structure. (Edited author abstract) 1 Ref.

Title: Probabilistic equivalent linear soil spring stiffness analysis for gravity platforms. Illustrative example.
Author: Kulatilake, Pinnaduwa H. S. W.; Lacasse, Suranne; Gabr, Mohammed
Corporate Source: Norwegian Geotechnical Inst, Oslo, Norw
Source: Computers and Geotechnics v 12 n 1 1991 p 29-54 Publication Year: 1991
Abstract: The companion paper suggests a procedure to perform equivalent linear soil spring stiffness analysis for gravity platforms founded on clay and loaded under undrained conditions. This paper provides an example to illustrate the suggested procedure. The CONDEEP SP4-10-24 platform concept proposed for the Troll field was used for the example. Plastic Drammen clay was chosen as the foundation soil to a depth of 150 m. Wave loads on the platform were estimated assuming a 70 yr platform life, plus an 18 hr storm build-up, plus a 6 hr storm with a return period of 100 yrs. The constitutive model for the soil was calculated for the 6 hr rare event in the load history, assuming that the rare even occurs at the end of the load history. The coefficients of variation of the equivalent soil spring stiffnesses (horizontal and rotational) were found to be around 0.50. (Author abstract) 19 Refs.

Title: Application of the hybrid frequency-time-domain procedure to the soil-structure interaction analysis of a shear building with multiple nonlinearities.
Author: Darbre, G. R.
Corporate Source: Swiss Div of Safety of Dams, Bern, Switz
Source: Soil Dynamics and Earthquake Engineering V First Int Conf Soil Dyn Earthquake. Publ by Computational Mechanics Publ, Southampton, Engl. p 441-454 Publication Year: 1991
Abstract: The hybrid frequency-time-domain procedure is applied to the nonlinear seismic analysis of a 6-story shear building interacting with the supporting soil. Both the frequency dependence of the foundation stiffness coefficients and the nonlinear hysteretic characteristics of the individual stories are retained in the analysis. The reliability of the analysis results is confirmed by way of comparison with the results of a time-stepping algorithm for the specialized case of constant soil-stiffness coefficients. The influence of the frequency dependence of the foundation stiffness coefficients on the seismic response is less important in the nonlinear case than in the linear case. For all practical purposes, it may be disregarded in the nonlinear case. (Author abstract) 8 Refs.

Title: 't-z' Approach for cyclic axial loading analysis of single piles.
Author: Chin, J. T.; Poulos, H. G.
Corporate Source: Univ of Sydney, Sydney, Aust
Source: Computers and Geotechnics v 12 n 4 1991 p 289-320 Publication Year: 1991
Abstract: A simple and efficient numerical approach is presented for the cyclic axial loading analysis of vertical single piles embedded in a layered soil profile. The soil medium along the embedded pile is represented by simple 't-z' curves which define the shear stress-vertical displacement response of the soil at each particular depth. A hyperbolic 't-z' representation of the soil medium presented by Chin and Poulos, which caters for the case of a two-layered and 'Gibson' soil profiles, is utilised. Under cyclic loading conditions, the well-known Masing's criteria governing the unloading and reloading responses are incorporated into the 't-z' curves. The cyclic loading effects of pile capacity degradation and accumulation of pile displacement are catered for in an approximate manner. Some numerical results are presented to show the important parameters affecting the cyclic response of single piles embedded in a layered soil. Finally, a comparison with field measurements of a cyclic pile load test shows general agreement between numerical and field results. (Author abstract) 38 Refs.

Title: Effect of seismic pore pressures on ground response.
Author: Vucetic, M.
Corporate Source: Univ of California, Los Angeles, CA, USA
Source: Deformation of Soils and Displacements of Structures X ECSMFE Proceedings of the International Conference on Soil Mechanics and Foundation Engineering v 2. Publ by A.A. Balkema, Rotterdam, Neth. p 873-876 Publication Year: 1991
Abstract: The effect of pore pressure development in deposits of saturated sands on the response of the soil profile to earthquake loading is analyzed. The analysis is performed using a computer model which incorporates a model for pore pressure buildup based on cyclic shear strains. The results show that cyclic shear strains are directly related to seismic pore pressure buildup and associated degradation of stiffness and strength of soil deposit. Their comparison with recorded seismic data also indicates that the maximum ground surface acceleration on top of a liquefied site can hardly exceed a value of approximately 0.2g. (Author abstract) 7 Refs.

Title: One-dimensional analysis of soil plugs in pipe piles.
Author: Randolph, M. F.; Leong, E. C.; Houlsby, G. T.
Corporate Source: Univ of Western Australia, Aust
Source: Geotechnique v 41 n 4 Dec 1991 p 587-598 Publication Year: 1991
Abstract: Under static loading in compression, open-ended piles may fail in a plugged mode, with the soil plug moving with the pile, or in an unplugged mode, with shear failure occurring between the soil plug and the pile shaft. It may be shown that, under drained loading conditions, the former mode of failure will generally occur, because arching action within the pipe pile leads to high frictional capacity of the plug. However, under faster rates of loading relevant to the offshore environment, the increase in effective stresses within the soil plug is limited and the plug capacity is significantly lower. The Paper presents a simple one-dimensional analysis of the soil plug under partially drained conditions. The analysis has been implemented numerically, and the resulting program used to derive design charts which give the plug capacity as a function of the soil plug parameters and the rate of loading. These design charts are presented in appropriate non-dimensional form, with example calculations included for typical offshore piles in calcareous soil. (Author abstract) 14 Refs.

Title: Seismic soil-pile interaction analysis of a marine piled pier.
Author: Gao, M.; Ding, W. N.; Fung, H. I.; Yu, J. F.
Corporate Source: Nanjing Hydraulic Research Inst, Nanjing, China
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 470-474 Publication Year: 1992
Abstract: In this paper, linear and nonlinear soil - pile interaction behavior is taken into consideration in the seismic safety analysis of marine piled structures. Comparisons between the results of linear and

nonlinear soil condition with the results from analysis without taking into consideration of the soil - pile interaction are made. (Author abstract) 11 Refs.

Title: Application of the hybrid frequency-time-domain procedure to the soil-structure interaction analysis of a shear building with multiple nonlinearities.
Author: Darbre, G. R.
Corporate Source: Swiss Div of Safety of Dams, Bern, Switz
Source: Soil Dynamics and Earthquake Engineering V First Int Conf Soil Dyn Earthquake. Publ by Computational Mechanics Publ, Southampton, Engl. p 441-454 Publication Year: 1991
Abstract: The hybrid frequency-time-domain procedure is applied to the nonlinear seismic analysis of a 6-story shear building interacting with the supporting soil. Both the frequency dependence of the foundation stiffness coefficients and the nonlinear hysteretic characteristics of the individual stories are retained in the analysis. The reliability of the analysis results is confirmed by way of comparison with the results of a time-stepping algorithm for the specialized case of constant soil-stiffness coefficients. The influence of the frequency dependence of the foundation stiffness coefficients on the seismic response is less important in the nonlinear case than in the linear case. For all practical purposes, it may be disregarded in the nonlinear case. (Author abstract) 8 Refs.

Title: Three dimensional soil-structure interaction analysis. Deformable structures in multilayered soil mass.
Author: Kundu, T.; Mathur, R. P.; Desai, C. S.
Corporate Source: Univ of Arizona, Tucson, AZ, USA
Source: Engineering Computations (Swansea, Wales) v 8 n 2 Jun 1991 p 153-180 Publication Year: 1991
Abstract: A new hybrid method based on three-dimensional finite element idealization in the near field and a semi-analytic scheme using the principles of wave propagation in multilayered half space in the far field is proposed for the dynamic soil-structure interaction analysis. The distinguishing feature of this technique from direct or indirect boundary integral techniques is that in boundary integral techniques a distribution of sources are considered at the near field boundary. Strengths of these sources are then adjusted to satisfy the continuity conditions across the near-field/far-field interface. In the proposed method unknown sources are placed not at the near field boundary but at the location of the structure. Then the Saint-Venant's principle is utilized to justify that at a distant point the effect of the structure's vibration can be effectively modelled by an equivalent vibrating point force and vibrating moment at the structure's position. Thus the number of unknowns can be greatly reduced here. For soil-structure interaction analysis by this method one needs to consider only three unknowns (two force components and one in-plane moment) for a general two-dimensional problem and six unknowns (three force components and three moment components) for a general three-dimensional problem. When a vertically propagating elastic wave strikes a structure which is symmetric about two mutually perpendicular vertical planes the structure can only vibrate vertically for dilatational waves and horizontally for shear waves. Under this situation the number of unknowns is reduced to only one whereas in boundary integral and boundary element techniques the number of unknowns is dependent on the number of nodes at the near field boundary, which is generally much greater than six. Several example problems are solved in this paper using this technique for both flexible and rigid structures in multilayered soil media. (Author abstract) 32 Refs.

Title: Effect of ground vibration induced by pile driving on adjacent structures.
Author: Rahimi, M. M.
Corporate Source: CSIRO, Aust
Source: Deformation of Soils and Displacements of Structures X ECSMFE Proceedings of the International Conference on Soil Mechanics and Foundation Engineering v 2. Publ by A.A. Balkema, Rotterdam, Neth. p 831-835 Publication Year: 1991
Abstract: The results of theoretical and experimental investigations on the effect of vibrations induced by drop hammer and cased driven piles on nearby structures are presented. Using a seismograph and power spectrum analysis, the crack width movement during and after piling period are analysed. Differential

crack movement at top and bottom of cracks due to densification of a silty sand layer are investigated. It was shown that the amplitude and frequency of vibrations increases with an increase in the bearing capacity of soil, and the maximum vibrations occur at knocking out of the plug from the end of the metal casing. It was also shown that crack movement stabilized a few weeks following piling, and residual crack movement depends on mechanical properties of soils, and soil-structure interaction. (Author abstract) 11 Refs.

Title: Seismic soil-pile interaction analysis of a marine piled pier.
Author: Gao, M.; Ding, W. N.; Fung, H. I.; Yu, J. F.
Corporate Source: Nanjing Hydraulic Research Inst, Nanjing, China
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 470-474 Publication Year: 1992
Abstract: In this paper, linear and nonlinear soil - pile interaction behavior is taken into consideration in the seismic safety analysis of marine piled structures. Comparisons between the results of linear and nonlinear soil condition with the results from analysis without taking into consideration of the soil - pile interaction are made. (Author abstract) 11 Refs.

Title: Linear and nonlinear dynamics of the troll gravity platform.
Author: Leira, Bernt J.; Karunakaran, Daniel; Svano, Geir; Skomedal, Eiliv
Corporate Source: SINTEF Structural Engineering, Trondheim, Norw
Source: Proc Second Int Offshore Polar Eng Conf. Publ by Int Soc of Offshore and Polar Engineerns (ISOPE), P.O.Box 1107, Golden, CO, USA. p 454-462 Publication Year: 1992
Abstract: Dynamic soil-structure interaction of the Troll gravity platform is considered. A model for nonlinear soil behaviour is presented, and corresponding parameters which are based on experimental investigations in conjunction with finite element soil modelling are presented. A procedure for stochastic dynamic response analysis based on a finite element model of the gravity platform is outlined. Response maxima in each short-term stationary sea-state are fitted to Weibull three-parameter distributions estimated from simulated response time series. Comparison is made between extreme response predicted from nonlinear versus linear analyses. Sensitivity of extreme response to soil stiffness is also investigated. (Author abstract) 10 Refs.

Title: Interaction soil-foundation-superstructure.
Author: Van Weele, A. F.
Corporate Source: Technological Univ Delft, Neth
Source: Deformation of Soils and Displacements of Structures X ECSMFE Proceedings of the International Conference on Soil Mechanics and Foundation Engineering v 3. Publ by A.A. Balkema, Rotterdam, Neth. p 1075-1081 Publication Year: 1991
Abstract: Foundation soils are nearly always more flexible than the foundation elements, buried or in contact with that soil, while the connections between the foundation elements and the superstructure are generally rigid. Deformations of the soil are partly created and partly resisted by the foundation elements, often resulting in complex stress distributions in the superstructure. The stress distribution in the subsoil is to be in harmony with the resulting deformations in it. So, an insight in the said interaction between foundation and superstructure can only be achieved if the deformations are under all circumstances given full attention. (Author abstract) 5 Refs.